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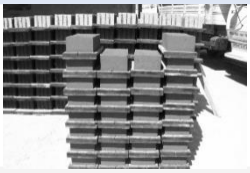

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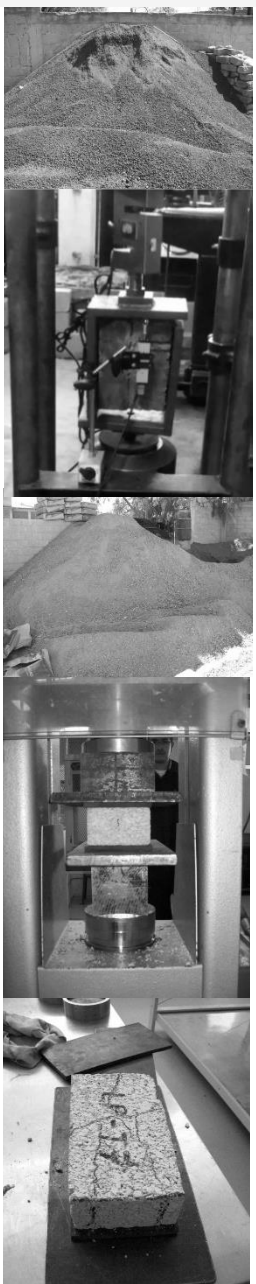

Journal of the Latin-American Association of Quality Control, Pathology and Recovery of Construction

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RAV7N1, January - April 2017

Message from the Editor in Chief and Guest Editor

JOURNAL OF THE LATIN-AMERICAN ASSOCIATION OF QUALITY CONTROL, PATHOLOGY AND RECOVERY OF CONSTRUCTION.

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With great satisfaction, we present the first number of the seventh year of the ALCONPAT journal.

The aim of the journal is to publish case studies within the scope of the Association, namely quality control, pathology and recovery of constructions, including basic and applied research, reviews and documentary research.

This edition presents our fifth Special Issue, this time dedicated to **smart applications for smart sustainability on maintenance and recovery of constructions** for celebrating the extense academic trajectory of Dr. Nemkumar Banthia and Dr. Muhammed Basheer that were honored during the 2nd R N Raikar Memorial International Conference, held at Mumbai, India.

The V7 N1 issue begins with a research from **India**, in which S. Bansal et al evaluated a two iconic bridges project over River Yamuna at Delhi. Both of them are being constructed and were reviewed from a sustainability point of view by using the Fuzzy-Vikor technique. In general, sustainable features are providing a balance between the natural environment and economy. During the study, it was realized that social, economic and environmental are the established parameters of sustainability for developed countries only whereas other issues like Governance, Technical parameters and Inner Engineering also play a key role for developing economies like India.

The second paper from **Canada** by R. Gupta and A. Biparva deals with the addition of crystalline water proofing admixtures as one method to decrease the permeability of concrete. Stage one was performed under ASTM specified conditions and a modified stage where more severe drying conditions than that described in the ASTM test standard were used. These modified conditions simulated inadequate curing under extreme exposure conditions as experienced by concrete in many parts of the world. The test results indicated that the water proofing admixtures can effectively reduce the early-age shrinkage cracking. The possible reasons for this secondary advantage of crystalline water proofing admixture is also hypothesized in this paper.

In the third paper from **India**, A. Narayanan and K. V. L. Subramaniam discuss how piezoelectric based PZT (Lead Zirconate Titanate) smart sensors offer significant potential for continuously monitoring the development and progression of internal damage in concrete structures. Changes in the resonant behavior in the measured electrical conductance obtained from electro-mechanical (EM) response of a PZT bonded to a concrete substrate was investigated for increasing levels of damage. Changes in the conductance resonant signature from EM conductance measurements were detected before visible signs of cracking. The root mean square deviation of the conductance signature at resonant peaks was shown to accurately reflect the level of damage in the substrate. The findings presented by the authors provide a basis for developing a sensing methodology using PZT patches for continuous monitoring of concrete structures.

The fourth paper, by A. Tena et al from **México,** presents a proposal to update the masonry index compressive strength design value f^*_m for solid concrete units for the masonry guidelines of Mexico´s Federal District Code. Solid units were made taking into account the characteristics of the most commonly used raw materials available in the Valley of Mexico. Different tests were conducted for both, raw materials and the obtained concrete units. Based upon test results, it is illustrated why it is much better to design masonry structures based upon the experimental data of the units to be used at the construction site rather than using index values proposed in building codes.

Another paper from K. van Breugel and T. A. van Beek from the **Netherlands**, deals with ageing as an inherent feature of concrete nature. Yet it seems to be a rather new topic in both science and engineering. The main reason for increasing attention for ageing as a topic is the growing awareness that, particularly in industrialized countries, ageing of our assets is a financial burden for the society and affects the overall sustainability of our planet. In this contribution, the authors address the urgency and challenges that represent ageing of concrete structures. The complexity of ageing problems was illustrated by watching in detail the evolution of concrete mix design and the consequences thereof for the long-term performance of concrete structures.

S. Vasudeo Mehendale et al from **India**, wrote the sixth paper where they remember that masonry is generally strong in compression and weak in tension. This situation can be improved by introducing reinforcement in bed joints of masonry. Strength of reinforced masonry is influenced by interfaces between brick, mortar and reinforcement in addition to the properties of brick, mortar and reinforcement. Experimental protocol has been defined to characterise the behaviour of reinforced brick masonry joint, with reinforcement steel embedded in cement mortar 1:6. Considering critical bond mechanisms, an attempt was made to put-forth a novel approach for development of a pseudo interface element representing three different materials (viz. brick, mortar and reinforcement) and two interfaces (reinforcement-mortar (RM) and brick-mortar (BM)). The developed pseudo interface element would help engineers to arrive at the most suitable and economical reinforced masonry solution.

In the seventh paper from **México**, L. M. Reynosa Morales et al present a study about emergy analysis where an environmental valuation method was applied to concrete mixing with the purpose of evaluating its dependence on non-renewable natural resources. The quantity of environmental resources used in production was measured in terms of equivalent solar energy. The resulting transformities were compared to show that emergy analysis is sensible to the local context and the limits of the reference system. The results obtained show that concrete mixing is highly dependent on external resources. Semi-industrialized concrete was found to be the most sustainable.

The last but not least paper from **Nepal** was authored by B. L. Nyachhyon. This is a documentary research to pay attention of local and international community including the government and donors to gear up for policy reform and create an environment for investing in proactive earthquake safety initiatives before the next earthquake strikes. The paper focuses on the outcome of the author's continuous interaction with local community since 1985 on the need for extended earthquake safety initiatives through stakeholders' easy access to technical assistance and financial resources. The most neglected aspect in the earthquake initiatives of Nepal was the lack of state ownership and dedicated responsible institutions. The opportunity created by the April 2015 earthquake is now a chance to apply those initiatives

We are confident that the papers in this special number will become a reference for those readers involved in cases and research related to properties, structures and durability of sustainable materials and structures. We are grateful to all authors of this special number for their endeavors towards the preparation of high quality papers in time.

It is noteworthy that the commitment and efforts from authors, reviewers, and editorial board during these past six years, have positioned the Alconpat journal within the Indexes of Conacyt (National Council of Science and Technology of Mexico), Latindex and Scielo.

On behalf of he Editorial Board

Pedro Castro Borges — Editor in Chief
Surendra Manjrekar — Guest Editor

COMMENTARY FOR THE SPECIAL ISSUE

BY

NEMKUMAR BANTHIA
AND
P. A. MUHAMMED BASHEER

Concrete is currently the most popular construction material in the world. This has been achieved through progress in research and developmental activities over many decades by numerous researchers and practioners. With the intention of sharing the latest developments in areas of innovation, research and practical application of cement and concrete in the construction industry, particularly to bring researchers, practitioners, specifiers, material developers and end users in one platform, the India Chapter of the American Concrete Institute started the R. N. Raikar International Memorial Conference series in 2012. The organizers also decided to felicitate an internationally leading researcher in any of the above fields. In 2012, the honoree was Prof Surendra Shah from North-western University, USA. We both were chosen to be the honorees during the second conference in December 2015 which was held at Mumbai. Therefore, it is our honor and pleasure to write this commentary on the R. N. Raikar International Memorial Conference.

Both of us have been participants of many international conferences globally and whilst comparing with them we are pleased to confirm that the organizers convened a truly international event as part of the second R. N. Raikar International Memorial Conference. We were extremely impressed with the quality of the papers, presentations and the arrangements. The conference provided an opportunity to discuss some of the recent developments in materials technology, performance of concrete in structures, investigation techniques and service life prediction. It was particularly pleasing to see papers on the practical application of recent developments in some global projects. Keynote lectures were given by eminent international researchers.

As the technical committee was very pleased with the quality of the papers, they have decided to publish a selection in international journals. This is to help achieve wider publicity of the recent developments reported at the conference. We believe readers will find these papers useful for their research or practical application of research, as the case maybe. We would like to thank all authors for their contributions to this special issue of Revista ALCONPAT. Further, we wish to express our gratitude to the organizing committee for felicitating us at the event.

Professor Nemkumar Banthia,
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Professor P.A. Muhammed Basheer
University of Leeds, Leeds, UK

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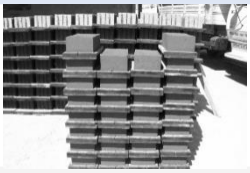

CONTENT

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Sustainability evaluation of two iconic bridge corridors under construction using Fuzzy Vikor technique: A case study

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ABSTRACT

Two iconic bridge projects over river Yamuna in Delhi under construction have been evaluated from sustainability criteria using Fuzzy-Vikor technique. The Barapulla elevated road project was more found to be more sustainable in comparison to the Signature bridge project in terms of various indicators identified during the study. In general, the goals of providing sustainable features are finding a balance between what is important to the community, to the natural environment and is economically sound. During the study, it was verified that social, economic and environmental are the established parameters of sustainability for developed countries only whereas other issues like governance, technical parameters and inner engineering also play a key role for developing economies like India.
Keywords: sustainability; Fuzzy-Vikor; governance; technical parameters; inner engineering.

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Evaluación de la sostenibilidad de dos corredores de puentes icónicos en construcción utilizando la técnica Fuzzy Vikor: Un estudio de caso

RESUMEN

Dos proyectos en etapa de construcción de puentes icónicos sobre el río Yamuna en Delhi han sido evaluados a partir de criterios de sostenibilidad utilizando la técnica de Fuzzy-Vikor. El proyecto de paso elevado de Barapulla resultó ser más sostenible en comparación con el proyecto del puente Signature en términos de varios indicadores identificados durante el estudio. En general, los objetivos de proporcionar características de sostenibilidad ofrecen un equilibrio entre lo que es importante para la comunidad, el medio ambiente natural y lo económicamente sólido. Durante el estudio se verificó que los parámetros sociales, económicos y ambientales son los parámetros establecidos de sostenibilidad para los países desarrollados, mientras que otros como la gobernanza, los parámetros técnicos y la ingeniería interna también desempeñan un papel clave para las economías en desarrollo como la India.

Palabras clave: sostenibilidad; Fuzzy-Vikor; gobernancia; parámetros técnicos; ingeniería interna

Avaliação de sustentabilidade de Duas pontes emblemáticas e em construção numa importante via aérea usando a técnica Fuzzy Vikor: Um estudo de caso

RESUMO

Dois projetos de pontes emblemáticas sobre o rio Yamuna em Delhi, em construção, foram avaliadas a partir de critérios de sustentabilidade usando a técnica de Fuzzy-Vikor. O Projeto de Via Elevada de Barapulla foi considerado o mais sustentável em comparação com o Projeto Signature Bridge em termos de vários indicadores identificados durante o estudo. Em geral, os objetivos de fornecer recursos sustentáveis são os de encontrar um equilíbrio entre o que é importante para a comunidade, para o ambiente natural e é economicamente sólido. Durante o estudo, percebe-se que o social, o econômico e o ambiental são os parâmetros estabelecidos de sustentabilidade para os países desenvolvidos enquanto outras questões como governança, parâmetros técnicos e engenharia interna também desempenham um papel fundamental para economias em desenvolvimento como a Índia.

Palavras chave: sustentabilidade; Fuzzy-Vikor; governança; parâmetros técnicos; engenharia interna.

1. INTRODUCTION

The idea of sustainability has been distinguished as a worldwide need and is most ordinarily characterized as "Improvement that addresses the issues of the present without trading off the capacity of future eras to address their own particular issues. This idea has infested whole ranges of Engineering, involving transportation frameworks building.

This Research task begins with depicting the eminent thinking on what constitutes sustainability of the transportation framework amid development and how to perform it. Further the study identifies some of the key transportation system sustainability issues through construction in the Metropolitan cities like Delhi. In this research, Sustainability indicators of the transportation corridor through development in an urban domain have been perceived and itemized out. The research has been made on Signature Bridge being constructed on river Yamuna by DTTDC (Delhi Tourism and Transportation Development Corporation Ltd.) and Barapulla Elevated

Corridor project being constructed by the PWD (Public Works Department). Amid the research study was made at both the sites in their construction period, and it was found that Sustainability of these transportation corridors while the development stage is just not restricted to just three Pillars, but rather in actuality much past that. Finally, the real center of study lies on showing a correlation between the afore mentioned two construction sites by two government organizations, that is PWD and DTTDC, under the identical urban environment, by utilizing the Fuzzy rationale strategy to assess sustainability taking into account the perceived sustainability pointers utilizing information collected by directing different reviews (survey proforma) from the field specialists and the general population (occupants/suburbanites). This research work obtains its motivation and guidance from similar project undertaken by Shishir Bansal et al. “Sustainability Indicators of a Transportation Corridor during Construction in an Urban Environment”.

This study is based on application of fuzzy technique. Fuzzy logic is referred to as a way of “reasoning with uncertainty.” It gives an all-around characterized system to manage dubious and not completely characterized information, so one can make exact findings from uncertain information the fuzzy theory provides a mechanism for representing linguistic constructs such as “many,” “low,” “medium,” “often,” “few.” Notions like rather tall or quick can be figured numerically and prepared with a specific end goal to apply a more human-like mindset in the programming. As a rule, the fuzzy rationale gives a surmising structure that empowers suitable human thinking capacities.

2. SELECTION OF SITE

Two iconic bridges of Delhi that are Signature Bridge and Barapulla elevated Corridor have been taken into consideration for sustainability review.

SIGNATURE BRIDGE AT WAZIRABAD: Signature bridge project or Wazirabad bridge project is an upcoming project of international significance. The Bridge over River Yamuna consists of a main bridge with eastern and western approaches and creation of tourist destination along the east and west banks.

BARAPULLA ELEVATED ROAD CORRIDOR: Elevated Road Project over Barapulla Nallah is a corridor connecting East and South Delhi. The Project has been conceived in three phases with nodal locations as Mayur Vihar in East Delhi and Aurobindo Marg in South Delhi with intermediate locations as Sarai Kale Khan and Jawahar Lal Nehru Stadium.

It was found out that both projects have striking similarities, which led to formation of common ground for unbiased comparison of sustainability. The afore mentioned similarities are as follows:

i. Both projects are iconic bridges: Signature Bridge is an asymmetric cable stayed bridge with main span of 251 m, while the Bridge over River Yamuna in Barapulla Phase III is Extra Dose bridge with multi spans of 120 m. In both the cases the deck is supported on Cables.
ii. Both projects are conceived on new alignments
iii. Both projects are carried out in phases where partially completed sites have been opened for public use
iv. Both projects were constructed in same period i.e. their construction works begin prior to commonwealth games of 2010
v. Both projects boast about usage of new and highly improvised technologies. Segmental constructions have been adopted in both projects.
vi. Both projects have their major portions constructed away from the urban parts of city and there has been least disturbance to the public. The normal life has not been hindered in any manner.

3. METHODOLOGY ADOPTED FOR THE RESEARCH

Following procedure has been followed in this research to identify the sustainability indicators.

i. Selection of a corridor under construction and defining the infrastructure criteria for the corridor.
ii. Developing sustainability indicator categories
iii. Identifying sustainability indicators
iv. Compiling a proforma that includes sustainability indicators and columns for rating
v. Assigning quantitative as well as qualitative ratings to the recognized indicators by furnishing ratings from the expert's opinions.

First, preliminary survey of the selected sites was carried out at different times during both day and night. Its main purpose was to identify certain issues which hinder the smooth movement of traffic and also those which are problematic in execution and protection of ongoing project. The list of 43 such issues was developed and then they were classified into six categories and each category is defined as Sustainability Indicators. For an Urban Environment and developing city like New Delhi, the triple bottom line concept of sustainability does not get fit. It requires extension to accommodate the local conditions. Accordingly, the triple bottom line concept is extended to six broad sustainability indicators. Based on the classification of these indicators, a questionnaire was framed and opinion of experts in this field from CRRI, PWD, BRO, Consultants, RITES etc. was obtained and with the opinion of experts, rating to these indicators was assigned based on Fuzzy methodology.

Table 1. Identified Sustainability Indicators

S. No.	SUSTAINABILITY INDICATORS
	A. ENVIRONMENTAL
1.	Air Pollution
2.	Existing Drainage system
3.	Noise pollution during day
4.	Noise pollution during night
5.	Depletion of Green Belt
6.	Plantation scheme
7.	Alternate schemes for make the project more sustainable
	B. SOCIAL
8.	Health of workers
9.	Welfare activities for family of workers
10.	Sanitation conditions
11.	First Aid facilities
12.	Safety measures
13.	Increase in stress level of residents/commuters
14.	Impact on Health of residents/commuters
15.	Impact on safety of residents/ commuters
16.	Preserving the social spaces like cremation ground, Sur Ghat
17.	Public attraction with the aesthetics of the Project
18.	Utility of the Project to Public
19.	Preserving the heritage structures

C. ECONOMICS	
20.	Increase in Travel time
21.	Increase in travel cost
22.	Disturbance to the business/Employment of nearby residents
23.	Increase in cost of Construction due to lack of funds
24.	Increase in cost of Construction due to time overrun
D. TECHNICAL	
25.	Display of Project Details
26.	Traffic Diversions
27.	Visibility and sight distance to moving traffic
28.	Lighting of Construction site
29.	Barricading the site
30.	Effectiveness of Technology used
31.	Handling of C & D Waste
32.	Quality Assurance on the Project
E. GOVERNANCE	
33.	Ensuring the mobility of Traffic within the project area by traffic Marshalls
34.	Maintenance of existing drainage system
35.	Maintenance of Barricades
36.	Maintenance of existing utilities
37.	Maintenance of existing greenery
38.	Time over run due to delay in Govt. decisions
39.	Time over run due to mismanagement at site
F. INNER ENGINEERING	
40.	Facilities of Yoga/meditation
41.	Performance of Rituals at site like Vishvakarma Puja, May Day
42.	Celebration during Festivals at site
43.	Motivation to workers by reward policy or otherwise

Based on Fuzzy theory, the ratings were assigned to these 43 indicators, as reflected in Table 1. In later stages a survey was conducted in commuters and residents nearby to evaluate the measures adopted by client and the construction agency in the form of questionnaire with rating scale of 0 to 9. Where 9 meant best arrangements and 0 signifies least arrangements causing maximum inconvenience.

4. FUZZY LOGIC

4.1 Preliminaries of Fuzzy Set Theory

Some related definitions of fuzzy set theory (Buckley 1985; Dubois and Prade 1987; Kaufmann and Gupta ,1991; Klir and Yuan, 1995; Pedrycz, 1994; Zadeh,1965) and Zimmermann (2001) are presented as follows.

4.1.1 Definition 1

A fuzzy set ã in a universe of discourse X is characterized by a membership function $\mu_{\tilde{a}}(x)$ that maps each element x in X to a real number in the interval [0, 1]. The function value $\mu_{\tilde{a}}(x)$ is termed the grade of membership of x in ã (Kaufmann and Gupta ,1991). The nearer the value of $\mu_{\tilde{a}}(x)$ is to unity, the higher the grade of membership of x is in ã.

4.1.2 Definition 2

A triangular fuzzy number (Fig. 1) is represented as a triplet $\tilde{a} = (a_1, a_2, a_3)$. Due to their conceptual and computation simplicity, triangular fuzzy numbers are very commonly used in practical applications (Klir and Yuan, 1995; Pedrycz, 1994). The membership function of $\mu_{\tilde{a}}(x)$ triangular fuzzy number is given by: $\mu_{\tilde{a}}(x) = 0,\ x \leq a1$, $\mu_{\tilde{a}}(x) = (x\text{-}a1)/(a_2\text{-}a_1)$, for $a_1 < x \leq a_2$ and $\mu_{\tilde{a}}(x) = (a_3\text{-}x)/(a_3\text{-}a_2)$, for $a_2 < x \leq a_3)$ and finally $\mu_{\tilde{a}}(x) = 0$, for $x > a_3$**,** where, a1, a2, a3 are real numbers and a1 < a2 < a3. The value of x at a2 gives the maximal grade of $\mu_{\tilde{a}}(x)$, i.e., $\mu_{\tilde{a}}(x) = 1$; It is the most probable value of the evaluation data. The value of x at a_1 gives the minimal grade of $\mu_{\tilde{a}}(x)$ i.e. $\mu_{\tilde{a}}(x) = 0$; It is the least probable value of the evaluation data. The narrower the interval $[a_1, a_3]$ is, the lower the fuzziness of the evaluation data is.

4.2 Linguistic variables and fuzzy set theory

In fuzzy set theory, conversion scales are used to transform the qualitative terms into fuzzy numbers. A scale of 0–9 is used to rate the criteria and the alternatives. Table 2 represent the conversion schemes for the qualitative, alternative and criteria ratings.

Table 2. Fuzzy transformation for qualitative criteria weightage and site ratings

Criteria weightage		**Site ratings**	
Qualitative Rating	Membership Function	Qualitative Rating	Membership Function
Very Low (VL)	(1,1,3)	Very por (VP)	(1,1,3)
Low (L)	(1,3,5)	Poor (P)	(1,3,5)
Medium(M)	(3,5,7)	Fair (F)	(3,5,7)
High (H)	(5,7,9)	Good (G)	(5,7,9)
Very High (VH)	(7,9,9)	Very good (VG)	(7,9,9)

4.3 VIKOR Method

In 1998 VIKOR (Vlse kriterijumska Optimizacija IKompromisno Resenje) method was developed by the Opricovic for the multi-criteria optimization of the complex systems. VIKOR method focuses on ranking and sorting a set of alternatives against various decision criteria assuming that compromising is only adequate to resolve conflicts. Alike some other MCDM methods like TOPSIS, VIKOR depends on an aggregating function that signifies closeness to the ideal, but unlike the TOPSIS, introduces the ranking index based on the particular measures of closeness to the ideal solutions and hence this method uses linear normalization for eliminating units of the criterion functions (Opricovic & Tzeng, 2004).

The VIKOR strategy was introduced as one appropriate method for actualizing within MCDM issue and was produced as a multi criteria choice for making a procedure to tackle a discrete decision making problem with non-commensurable and clashing criteria. This method focuses on the ranking and selection from a set of alternatives, and evaluates the compromise solution for a problem within conflicting criteria, which can aid the decision makers to reach a final solution. The multi-criteria measure for bargain positioning is produced from the LP–metric utilized as a totaling capacity as a part of a trade off programming method.

Assuming that each alternative is evaluated according to each criterion function, the compromise ranking could be performed by comparing the measure of closeness to the ideal alternative. The various m alternatives are denoted as $A_1, A_2, \ldots\ldots A_m$. For alternative A_i, the rating of the j^{th} aspect is denoted by f_{ij} (i= 1,2,…. m; j=1,2,… n), i.e., f_{ij} is the value of j^{th} criterion function for the alternative A_i, n is the number of criteria.

The compromise ranking algorithm of the VIKOR method has the following steps:

Step 1: To Assign ratings to various alternatives sites and criteria by decision makers (K Nos.) and experts (L Nos.)
Let us take a set of ***m*** alternatives sites called **A** = {A1, A2, ..., A_m} which we need to evaluate against a set of ***n*** criteria, that is **C** = {C1, C2, ..., Cn}.

(a) The criteria weights as assessed by experts are represented by $\mathbf{w_j}$ where (j=1,2, ..., n). The rating of each expert $\mathbf{E_l}$ (l = 1,2, …, L) for each criteria $\mathbf{C_j}$ (j= 1,2, .., n) are denoted by :
$(\mathbf{a_{jl}}, \mathbf{b_{jl}}, \mathbf{c_{jl}})$, where j = 1, 2, …, n; l= 1,2, ..., L;

(b) The performance ratings by the decision maker $\mathbf{D_k}$ (k = 1,2, …, K) for each alternative $\mathbf{A_i}$ (i=1,2, ..., m) according to criteria $\mathbf{C_j}$ (j= 1,2, ..., n) are denoted by:
$(\mathbf{a_{ijk}}, \mathbf{b_{ijk}}, \mathbf{c_{ijk}})$, where j = 1, 2, …, n; i= 1, …., m; k =1, 2, ..., K

Step 2: To compute the aggregate crisp ratings ($\mathbf{w_j}$) for each criteria by experts ans $\mathbf{D_k}$ corresponding to each criteria for alternatives and criteria.

The aggregated fuzzy weights ($\mathbf{w_{ij}}$) corresponding to each criterion are calculated as $w_j = (w_{j1}$; wj2; wj3) where

$$w_{j1} = \min\{w_{jl1}\},\ w_{j2} = \frac{1}{l}\sum_{l=1}^{L} w_{jl2},\ w_{j3} = \max\{w_{jl3}\} \quad (1)$$

W = (w_1, w_2… w_n) corresponding to each of the 'n' criteria

Crisp rating $w_j = (w_{j1} + 4{*}w_{j2} + w_{j3})/6$

Similaly aggregated fuzzy rating for each of the alternative m sites is computed.
$R_k = (a_k, b_k, c_k)$, where k=1,2,...,K, then the aggregated fuzzy rating is defined by R=(a, b, c), k=1,2,...,K where;

$$a=\min\{a_k\},\quad b = \frac{1}{K}\sum_{k=1}^{k} b_k,\quad c = \max\{c_k\} \quad (2)$$

Step 3: To compute the fuzzy decision matrix for 'K' decision makers, 'm' alternative sites and 'n' number of criteria

The fuzzy decision matrix (D) for the criteria (C_j) and the alternatives (A_i) is constructed as follows:

$$D = \begin{array}{c c} & \begin{array}{cccc} A_1 & A_2 & \ldots.. & Am \end{array} \\ \begin{array}{c} C_1 \\ C_2 \\ \ldots \\ C_n \end{array} & \begin{bmatrix} X_{11} & X_{12} & \ldots.. & X_{1m} \\ X_{21} & X_{22} & \ldots.. & X_{2m} \\ \ldots & \ldots & \ldots.. & \ldots \\ X_{n1} & X_{n2} & \ldots.. & X_{nm} \end{bmatrix} \end{array},\ i= 1, 2,...,m;\quad j= 1,2,\ldots,n \ (3)$$

Step 4: To defuzzify the elements of the fuzzy decision matrix corresponding to the alternatives and the criteria weights into crisp values.

a fuzzy number a~= (a_1, a_2, a_3) can be converted into a crisp number a by employing the below equation:

$$a = (a_1 + 4a_2 + a_3)/6 \quad (4)$$

Step 5: To Determine the best and worst values of critera rating where f_j* is best and values f_j^- is worst value

$$f_j^* = \max_i \{x_{ij}\} \quad (5)$$
$$f_j^- = \min_i \{x_{ij}\}$$

Step 6: To compute the values of S_i and R_i using the equations given below

$$Si = \sum_{j=1}^{n} w_j \frac{f_j^* - x_{ij}}{f_j^* - f_j^-} \quad (6)$$

$$Ri = max_j \; w_j \frac{f_j^* - x_{ij}}{f_j^* - f_j^-} \quad (7)$$

Step 7: To compute the values of Q_i using

$$Qi = v \frac{s_i - s^*}{s^- - s^*} + (1-v) \frac{R_i - R^-}{R^- - R^*} \quad (8)$$

where S* = minimum S_i, S^- = maximum S_i, R* = minimum R_i, and R^- = maximum R_i and v is the weight for the strategy of maximum group utility and here it is taken as 0.5

Step 8: To rank the alternatives by sorting the values Q, R and S in ascending order.

Step 9: To propose a compromise solution for the alternative ($A^{(1)}$) which is the best ranked by the measure Q(minimum) if the following two conditions are satisfied.

C1: Acceptable advantage

$$\text{If } Q(A^{(2)}) - Q(A^{(1)}) \geq DQ \quad (9)$$

Where $A^{(2)}$ is the alternative that holds second position in the ranking list according to Q and

$$DQ = 1/J\text{-}1, \text{ where j is number of criteria} \quad (10)$$

C2: Acceptable stability in decision making

The alternative $A^{(1)}$ should also be the best ranked by R or/and S. The settlement solution is stable only within a specific decision making process, and that could be the strategy of maximum group utility (when v>0.5 is needed), or ―by consensus when v = 0.5, or ―with veto ie (v<0.5). If one of the above conditions is not satisfied, then a set of settlement solutions is proposed, which consists of:

- Alternatives A (1) and A (2) if only the condition C2 is not satisfied Or
- Alternatives A (1), A (2), ... A(M) if the condition C1 is not satisfied;

A(M) is determined by the relation
Q(A(M)) - Q(A(1)) < DQ for maximum M (the position of these alternatives are in closeness).

5. NUMERICAL APPLICATION OF FUZZY LOGIC

In this section sustainability evaluation of the two transportation corridors as alternative sites namely A1 and A2, in Delhi, under construction have been carried out using the Fuzzy VIKOR technique. These project sites are Barapulla Elevated Corridor (A1) constructed by PWD and Signature Bridge (A2) constructed by DTTDC.
A committee of 10 experts (E1, E2… E10) was formed to obtain the qualitative ratings for the criteria and the alternatives.

Table 3. Qualitative assessments and aggregate fuzzy criteria ratings

Criteria	Qualitative rating										Aggregate Fuzzy weight	Crisp rating (W_j)
	E1	E2	E3	E4	E5	E6	E7	E8	E9	E10		
C1	VH	VH	VH	H	H	VH	VH	H	VH	VH	(5,8.4,9)	7.93
C2	H	VH	M	VH	M	M	H	VH	VH	H	(3,7.2,9)	6.80
C3	M	H	H	H	L	H	M	M	M	M	(1,5.6,9)	5.40
C4	H	VH	VH	VH	VH	H	H	H	H	VH	(5,8,9)	7.67
C5	VH	VH	M	H	H	VH	VH	H	H	H	(3,7.6,9)	7.07
C6	VH	VH	VH	H	H	M	H	M	H	H	(3,7.2,9)	6.80
C7	H	M	M	VH	H	H	VH	VH	H	H	(3,7.2,9)	6.80
C8	VH	VH	H	VH	H	VH	H	H	H	VH	(5,8,9)	7.67
C9	VH	H	L	VH	H	H	H	M	H	H	(3,6.8,9)	6.53
C10	VH	VH	H	VH	H	VH	H	H	H	H	(5,7.8,9)	7.53
C11	VH	VH	VH	VH	H	VH	VH	VH	H	VH	(5,8.6,9)	8.07
C12	VH	VH	VH	VH	VH	VH	VH	VH	VH	VH	(7,9,9)	8.67
C13	H	VH	VL	VH	M	VH	VH	H	M	VH	(1,7,9)	6.33
C14	VH	VH	VH	VH	M	VH	VH	H	H	H	(3,8,9)	7.33
C15	VH	VH	VH	VH	H	VH	VH	VH	VH	H	(5,8.6,9)	8.07
C16	H	VH	M	M	H	VH	VH	H	VH	H	(3,7.4,9)	6.93
C17	M	H	L	H	H	VH	M	M	H	M	(1,6,9)	5.67
C18	VH	VH	M	H	M	H	VH	VH	H	VH	(3,7.6,9)	7.07
C19	VH	M	M	M	M	H	VH	VH	VH	H	(3,7,9)	6.67
C20	VH	VH	VH	VH	M	VH	H	H	VH	H	(3,8,9)	7.33
C21	VH	VH	VH	VH	M	M	H	H	VH	H	(3,7.6,9)	7.07
C22	H	H	VH	VH	L	M	H	H	H	M	(1,6.8,9)	6.53
C23	H	H	H	VH	VH	VH	H	H	VH	VH	(5,8,9)	7.67

C24	H	H	H	VH	VH	VH	H	H	VH	VH	(5,8,9)	7.67
C25	H	H	M	H	L	L	H	M	VH	L	(1,5.6,9)	5.40
C26	VH	VH	VH	VH	VH	VH	VH	VH	VH	H	(5,8.8,9)	8.20
C27	VH	VH	H	VH	M	VH	VH	VH	H	H	(3,8,9)	7.33
C28	VH	VH	H	VH	VH	VH	VH	VH	VH	H	(5,8.6,9)	8.07
C29	VH	VH	H	VH	H	VH	VH	H	VH	VH	(5,8.4,9)	7.93
C30	VH	H	H	H	M	M	M	H	VH	H	(3,6.8,9)	6.53
C31	H	H	M	VH	H	VH	VH	H	H	H	(3,7.4,9)	6.93
C32	VH	H	VH	VH	VH	VH	VH	VH	H	H	(5,8.4,9)	7.93
C33	VH	VH	VH	VH	VL	VH	H	VH	VH	VH	(1,8,9)	7.00
C34	VH	VH	H	VH	H	VH	H	VH	VH	VH	(5,8.4,9)	7.93
C35	H	M	H	VH	H	VH	H	VH	H	M	(3,7.2,9)	6.80
C36	VH	H	VH	VH	VH	VH	H	VH	H	H	(5,8.2,9)	7.80
C37	VH	VH	M	VH	H	H	H	H	H	VH	(3,7.6,9)	7.07
C38	H	H	VH	VH	VH	H	H	M	VH	VH	(3,7.8,9)	7.20
C39	H	H	VH	VH	M	M	VH	VH	VH	H	(3,7.6,9)	7.07
C40	M	M	M	H	VL	M	VL	L	VL	L	(1,3.4,9)	3.93
C41	VL	L	H	VH	VH	M	VL	M	H	L	(1,5,9)	5.00
C42	M	VL	VH	H	VL	M	VL	M	M	VL	(1,3.8,9)	4.20
C43	VH	VH	H	H	VH	VH	H	H	VH	VH	(5,8.2,9)	7.80

The qualitative ratings into fuzzy triangular numbers and then we generate aggregate ratings using the equation (1). The following Table presents the aggregate fuzzy decision matrix for the both the alternative sites.

Generate aggregate crisp ratings for both the alternative sites using equation (4). Based on these values, we will calculate the best f_j* and the worst f_j- values of all 43 criteria using equation (5)

Table 4. The best values fj* and the worst values fj- of the 43 criteria

Criteria	Crisp Rating		Worst Value F_j^-	Best Value F_j^*
	A1 (PWD)	A2 (DTTDC)		
C1	6.15	6.01	6.01	6.15
C2	6.17	6.15	6.15	6.18
C3	6.28	6.17	6.17	6.28
C4	6.32	6.19	6.19	6.32
C5	6.83	6.53	6.53	6.83
C6	4.44	4.57	4.44	4.57
C7	6.75	6.59	6.59	6.75
C8	6.15	6.01	6.01	6.15
C9	4.33	4.52	4.33	4.52
C10	4.36	4.57	4.36	4.57
C11	6.8	6.53	6.53	6.85
C12	6.15	6.01	6.01	6.15
C13	6.09	6.12	6.09	6.12

C14	6.64	6.59	6.59	6.64
C15	6.8	6.51	6.51	6.8
C16	6.64	6.53	6.53	6.64
C17	6.15	6.01	6.01	6.15
C18	6.85	6.51	6.51	6.85
C19	6.85	6.53	6.53	6.85
C20	6.93	6.61	6.61	6.93
C21	6.85	6.53	6.53	6.85
C22	6.83	6.53	6.53	6.83
C23	6.85	6.56	6.56	6.85
C24	6.75	6.51	6.51	6.75
C25	6.83	6.53	6.53	6.83
C26	6.83	6.59	6.59	6.83
C27	6.83	6.51	6.51	6.83
C28	6.85	6.59	6.59	6.85
C29	6.83	6.53	6.53	6.83
C30	6.88	6.56	6.56	6.88
C31	6.85	6.53	6.53	6.85
C32	6.64	6.53	6.53	6.64
C33	6.64	6.56	6.56	6.64
C34	6.59	6.53	6.53	6.59
C35	6.83	6.56	6.56	6.83
C36	6.61	6.56	6.56	6.61
C37	5	5.72	5	5.72
C38	6.85	6.53	6.53	6.85
C39	6.59	6.53	6.53	6.59
C40	6.64	6.56	6.56	6.64
C41	6.64	6.56	6.56	6.64
C42	6.44	6.15	6.15	6.44
C43	6.64	6.51	6.51	6.64

Following table presents the values of Si, Ri and Qi for the two alternatives calculated using equations (6) - (8). The values of $S^*= 0.736$, $S^- = 5.76$, $R^*= 0.163$, $R^- =0.188$ are computed using equation (9).

Table 5. Values of Si, Ri and Qi

	A1(PWD)	A2(DTTDC)
Si	0.74	5.75
Ri	0.16	0.19
Qi	0	0

Table 6 ranks the two alternatives, by sorting the values of Si, Ri and Qi obtained from Table 5 in the ascending order. It can be seen from the above results as presented in Table 6 that site 1 that is Barapulla Elevated Corridor by the PWD is the best ranked by the measure of least value of Q_i. Therefore we now cross-examine it for the given two conditions those have been earlier discussed.

Table 6. Ranking the alternatives

S_i	A1	A2
R_i	A1	A2
Q_i	A1	A2

1). C1: acceptable advantage i.e. equation 9
Using equation 9 DQ = 1/43-1 = 1/42 = 0.0238.
Now to satisfy the condition $Q(A^{(2)}) – Q(A^{(1)}) \geq DQ$,where $A^{(1)}$ is the best ranked by the measure Q(minimum) and in our case it is A1
We have
Q(A2) - Q(A1) = 1 – 0 = 1 > 0.0238, hence the condition $QA^{(1)} – QA^{(2)} \geq DQ$ is satisfied.

2). C2: Acceptable stability in decision making using equation 10
Since site A1 is best ranked by S_i and R_i (considering the ―"by consensus rule v =0.5"), therefore it is declared to be as a more sustainable corridor.

6. RESULTS AND DISCUSSIONS

6.1 Results

The Fuzzy VIKOR technique was applied for sustainability evaluation of two major transportation corridors under construction i.e. (A1, A2) in New Delhi city. These projects were Barapulla Elevated Corridor being constructed by PWD (A1) and Signature Bridge being constructed by DTTDC (A2). The Final outcomes after the numerical application of Fuzzy VIKOR method exhibit that the site A1, i.e Barapulla Elevated Corridor being constructed by PWD is found to be more sustainable under the given conditions and the identified sustainability indicators

6.2 Discussions

The five-step methodology defined in this research can be used for any transport corridor to develop sustainability indicators. The five steps are

i. Selection of a corridor under construction and defining the infrastructure criteria for the corridor
ii. Developing the sustainability indicator categories
iii. Identifying the sustainability indicators
iv. Compilation of a proforma that include sustainability indicators and corresponding columns for rating
v. Assigning the quantitative as well as qualitative ratings to the recognized indicators by furnishing the ratings from the field expert's opinions

Each of these steps can be applied to evaluate a sustainable transportation corridor through construction in an urban environment. This process began with the requisite for categorization of the sustainability from its existing three pillars i.e. Economic, Social and Environmental aspects and excelled with the development of three more vital categories namely Governance, Technical and Inner Engineering. In later stages the individual parameters/indicators under these 6 sustainability categories were recognized by visiting the corridors through construction and consultation with the field experts. Finally, the process completed with the compilation of a proforma that furnishes Qualitative as well as Quantitative ratings to each identified sustainability indicator from the experts.

7. CONCLUSIONS

Following conclusions are drawn from the above study:

i. Through this research study it has been furnished that sustainability is not only based on three parameters but also depend on various other indicators that has been identified as per study.
ii. Various Sustainability Indicators through the construction stage has been identified for an elevated transportation corridor and hence are classified under various categories as covered in this research.
iii. The three pillars of sustainability namely social, economic and environmental are viable only for developed countries whereas in developing economies like India, where various other factors such as exponential increase in population etc., come into play, the need to introduce additional parameters arises.
iv. The comparative study of 2 iconic transportation corridors through construction, Barapulla Elevated Corridor being constructed by PWD (A1) and Signature Bridge being constructed by DTTDC (A2) has defined a methodology for future sustainability studies
v. The results of this study yield that Barapulla Elevated corridor is more sustainable as compared to the Signature Bridge.

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Do crystalline water proofing admixtures affect restrained plastic shrinkage behavior of concrete?

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ABSTRACT

This paper describes the effect of crystalline water proofing admixtures on early-age cracking in concrete. The performance of three different types of these admixtures was compared to that of control. This study has been performed in two stages. Stage one was performed under ASTM specified conditions and a modified stage where more severe drying conditions than that described in the ASTM test standard were used. These modified conditions simulated inadequate curing under extreme exposure conditions as experienced by concrete in many parts of the world. The test results indicate that the water proofing admixtures can effectively reduce the early-age shrinkage cracking. The possible reasons for this secondary advantage of crystalline water proofing admixture is also hypothesized in this paper.

Keywords: restrained plastic shrinkage cracking; crystalline water proofing admixtures; crack reduction ratio; time to first crack.

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¿Los aditivos de impermeabilización cristalina afectan al comportamiento de retracción plástica restringida del concreto?

RESUMEN

Este artículo describe el efecto de las mezclas cristalinas de impermeabilización sobre el agrietamiento en concreto a temprana edad. El rendimiento de tres tipos diferentes de estos aditivos se comparó con el de control. Este estudio se ha realizado en dos etapas: la primera se realizó bajo condiciones especificadas por ASTM y la segunda fue una modificación de la primera, en la que se usaron condiciones de secado más severas que las descritas en la norma de ensayo ASTM. Estas condiciones modificadas simulaban un curado inadecuado en condiciones extremas de exposición como las experimentadas por el concreto en muchas partes del mundo. Los resultados de la prueba indican que los aditivos de impermeabilización cristalina pueden reducir eficazmente el agrietamiento por retracción a edad temprana. Las posibles razones de esta ventaja secundaria son también supuestas en este trabajo.

Palabras clave: agrietamiento por retracción plástica restringida; mezclas cristalinas de impermeabilización; relación de reducción de grietas; tiempo para la primera grieta.

As adições cristalinas impermeabilizantes afetam o comportamento da retração plástica do concreto?

RESUMO

Este artigo descreve o efeito de aditivos de impermeabilização cristalina na fissuração precoce em concreto. O desempenho de três diferentes tipos destas adições foi comparado com o de controle. Este estudo foi realizado em duas etapas. A fase 1 foi realizada sob as condições especificadas na ASTM e uma fase modificada onde foram utilizadas condições de secagem mais severas do que as descritas na norma de ensaio ASTM. Estas condições modificadas simulam uma cura inadequada em condições extremas de exposição, tal como experimentado pelo concreto em muitas partes do mundo. Os resultados do ensaio indicam que as adições impermeabilizantes podem efetivamente reduzir as fissuras de retração. As possíveis razões para esta vantagem secundária da adição impermeabilizante cristalina também são discutidas neste artigo.

Palavras-chave: fissuração por retração plástica; adições cristalinas impermeabilizantes; relação de redução de fissuras; tempo para a primeira fissura.

1. INTRODUCTION

The demand for cement around the world has continued to be strong over the last decade even though there have been major concerns about the CO_2 emissions associated with its production. To make concrete more sustainable, recent measures being taken include use of limestone blended cement (recently launched in the Canadian market (Holcim, 2011)), use of alternate fuels (Vaccaro, 2006) and improved energy management to fire kilns, and use of higher amounts of SCMs (supplementary cementing materials) in concrete. A complementary approach to make concrete a sustainable material is to improve its service life by improving its durability. One of the key parameters that affects durability of concrete structures is its permeability. Various commercial methods available to decrease permeability of concrete have been previously discussed by the authors (Biparva & Gupta, 2010). Even though the main motivation of such crystalline admixtures is to make concrete less permeable over time, it is well known that these admixtures also modify the early-age properties of concrete. Crystalline Admixtures are one of the Permeability Reducer

Admixtures (PRAs) type as described by the American Concrete Institute (ACI) Committee 212. Contrary to hydrophobic or water-repellent materials, these products are hydrophilic which makes them to react easily when moisture enters into the pores/cracks of concrete. After taking this reaction in place, the admixture forms water insoluble pores/cracks blocking crystals that create very low permeable concrete due to increase in density of Calcium Silicate Hydrate (CSH, main cement hydration product) and higher resistance to water penetration. The matrix component which reacts is tri-calcium silicate (C_3S) and presence of water is also essential for the reaction. Depending on crystalline promoter and a precipitate formed from calcium and water molecules, active chemicals contained in cement and sand form these products. As a result of crystalline depositions into concrete matrix, pressure resistance of modified matrix increases as high as 14 bars. In this previously published work, the authors have discussed the various advantages of using a hydrophilic crystalline water proofing admixtures in concrete over other methods to make concrete water proof. They mention that by using crystalline waterproofing not only can concrete permeability be reduced, but also other properties such as self-sealing and shrinkage will be affected. Some results are summarized in this published paper, but a more focused research is required to investigate the effects of different integral waterproofing admixtures on key properties such as self-sealing and shrinkage. Most of the available literature only describes the effects of these admixtures on the permeability of concrete (Geetha & Perumal 2011), but key properties such as self-sealing and shrinkage are still not understood.

The effect of various admixtures, mineral additives, and fibers on restrained plastic shrinkage has been studied by many researchers. This includes studying the effect of shrinkage reducing admixtures (Weiss and Shah, 2002; Lura et al, 2007; Bentur et al, 2001), silica fume (Bloom and Bentur, 1995), limestone (Corinaldesi & Moriconi, 2009), fly ash (Gupta et al 2010), and fibers (Soroushian et al, 1992; Soroushian & Ravanbakhsh, 1998; Corinaldesi & Moriconi, 2009; Gupta et al, 2010) on restrained plastic shrinkage of concrete. However, a literature search by the authors to identify the effect of crystalline waterproofing admixtures on restrained plastic shrinkage using the ASTM C1579 test method did not result in any articles. Moreover, no previous studies reporting the effect of these water proofing admixtures on drying shrinkage could also be identified. The research study presented in this paper was initiated due to the lack of understanding about the effect of these water proofing admixtures on restrained plastic shrinkage potential of concrete. Research was conducted according to ASTM C1579 to study the effect of three different types of admixtures on shrinkage cracking. The test conditions specified in the test standard were modified to simulate more severe curing conditions that concrete is exposed to in many parts of the world.

2. EXPERIMENTAL PROCEDURE

2.1. Mix design

A control mix with target strength commonly specified in practice of 40 MPa was chosen for this study. To study the effect of crystalline based waterproofing admixtures on plastic shrinkage, three commercially available products were used to modify the control mix. The admixtures chemically reacts with infiltrated water/moisture in the pores/cracks of concrete and results in growth of microscopic crystals that block the flow of moisture, hence making concrete more impermeable. Dosages recommended by the manufacturers were used to modify the control mix. Admixture K, P, and X were added at 2.0%, 0.8%, and 1% of the cement mass respectively in the control mix. A w/c ratio of 0.55 was chosen for all the mixes. Mix proportions can be seen on Table 1.

Table 1. Mix proportions

Ingredient	Quantity (kg/m^3)
Portland cement	340
Gravel	1120
Sand	820
Water	187

2.2. Test set-up

Molds were used according to ASTM C 1579-06, to induce a crack along the center of the slab. Slabs were 355 (± 10) mm x 560 (± 10) mm x 100 (± 5) mm and contained a metal stress riser plate that was bolted to the bottom of the mold. The stress risers were used to induce a crack in the concrete at an early age. For each mix, two specimens were tested in the environmental chamber shown in Figure 1.

Figure 1. Instrumented environmental chamber for conducting restrained plastic shrinkage tests

Testing was done following ASTM C1579 standard in terms of materials, molds and specified environmental conditions. However, additional trials using higher temperatures and lower humidity were conducted to simulate conditions more severe than those specified by the test standard. According to the ASTM standard the temperature must be 36 ± 3 °C, the relative humidity 30 ± 10% and a minimum wind speed of 4.7 m/s over the center of the sample. The measured evaporation rate in the environmental chamber was greater than 1.0 kg/m^2/hr which is specified in the test standard.

2.3. Environmental Conditions

The conditions of the Environmental Chamber were regulated using a temperature and relative humidity controller. The temperature and relative humidity were recorded using a HOBOware data logger. A dual temperature and humidity sensor was placed above the center of each slab. Readings were taken by the logger every 10 seconds and the results were plotted using HOBOware software.

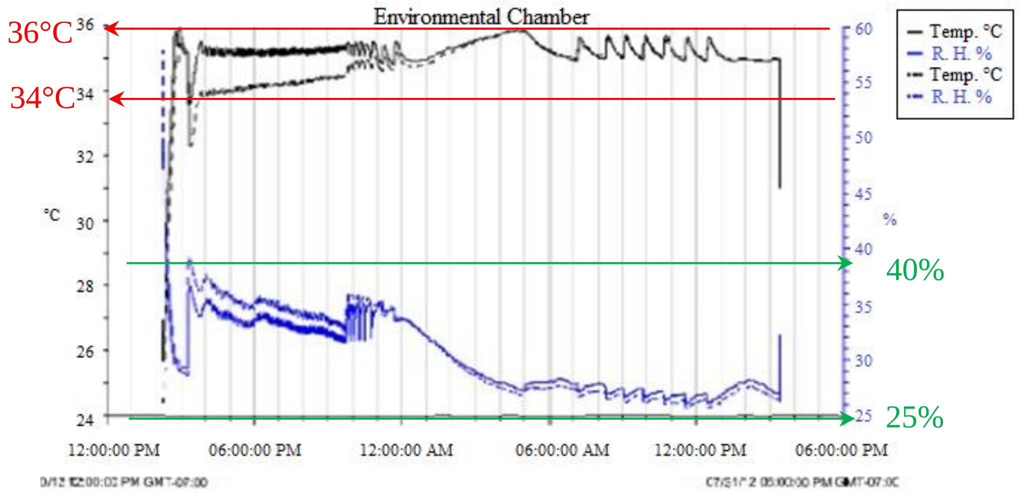


Figure 2. Typical screenshots of temperature and humidity vs. time (Standard conditions)

The graph above is a screen shot of the data and hence does not have very high resolution. This graph is however described below for clarity. Figure 2 shows a conditioning period of one hour when the temperature and relative humidity are brought up from ambient to standard test conditions. This is indicated by the first rise in temperature and drop in humidity. Later, there is a sharp increase in humidity and decrease in temperature, which indicates the slab being placed in the chamber. A few minutes after placing the specimens, both the temperature and humidity stabilize within the ASTM specified tolerances. The solid and dashed black curves represent the temperature at the top and bottom of the chamber respectively. Similarly, the two blue curves correspond to humidity values at the top and bottom of the chamber. Since the screen shots presented here have poor resolution, red lines and green lines have been added to allow reading the upper and lower bound temperature and humidity values respectively. Testing is conducted for 24 hours, after which the sample is removed and its crack size is measured. Once the specimens were placed in the environmental chamber the temperature was between 34°C and 36°C throughout the test (red lines), whereas the humidity was within a tight range of 25-40%. These values are within the limits allowed by ASTM.

2.3.1. Modified Condition

To study the behavior of the mixes tested previously under more severe drying conditions, the conditions in the environmental chamber were modified. As described earlier the solid red and blue lines have been added to read upper and lower bound temperature and humidity values respectively during the first 8 hours of the test. The dotted lines correspond to the modified conditions in the chamber after the first 8 hours.

As with the standard condition there is a conditioning period before placing the slab and a rebound period after. The modified condition was intended to expose specimens to more extreme temperature and humidity conditions. Conditions specified by the ASTM standard were used for the first 8 hours to emulate a work day were the concrete is monitored and maintained. After the 8 hour period the conditions were altered so that the temperature was gradually increased to 46±1°C. This resulted in the humidity to drop to a 15-27% range within 4 hours of these conditions being imposed. This approximately represented a 30% increase in the average temperature and a decrease of about 25% in the humidity. Testing under these conditios was also conducted for 24 hours, after which the cracks were measured.

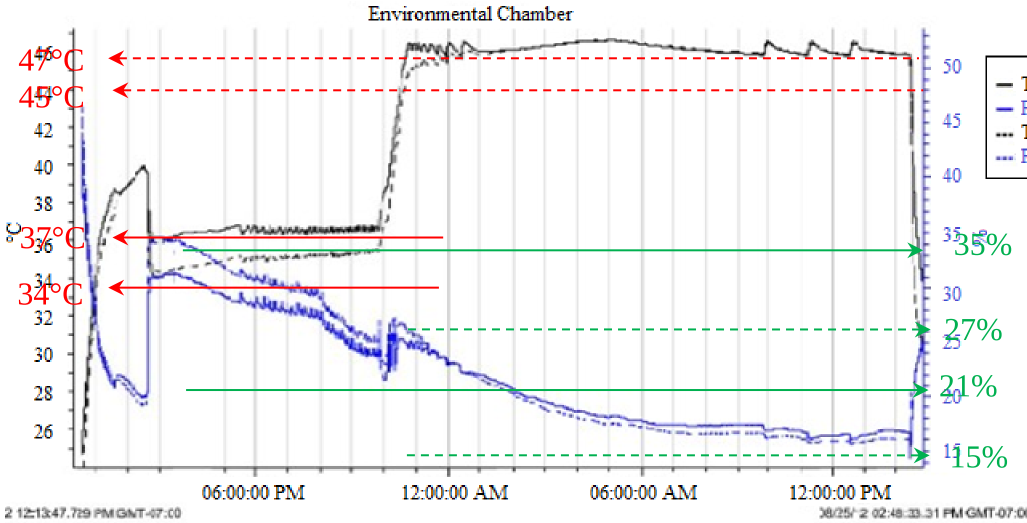


Figure 3. Typical screenshots of temperature and humidity vs. time (modified conditions)

2.4. Time to cracking

Even though not a requirement of the test standard, the time to first crack was measured using two HP AutoFocus 720i high definition video cameras. Video of the slab during the test was recorded to determine the time of cracking. This eliminated the need to manually observe the specimens for cracking.

3. RESULTS

3.1. Crack width

After 24 hrs, the test specimens were removed from the chamber and the cracks characterized. The crack size was measured using an optical handheld microscope at intervals of 10 mm along the length of the crack. The average of all the readings for the two specimens are presented in Table 2. Also, included in this table is the maximum crack width recorded in the specimens.

Table 2. Crack width measurements

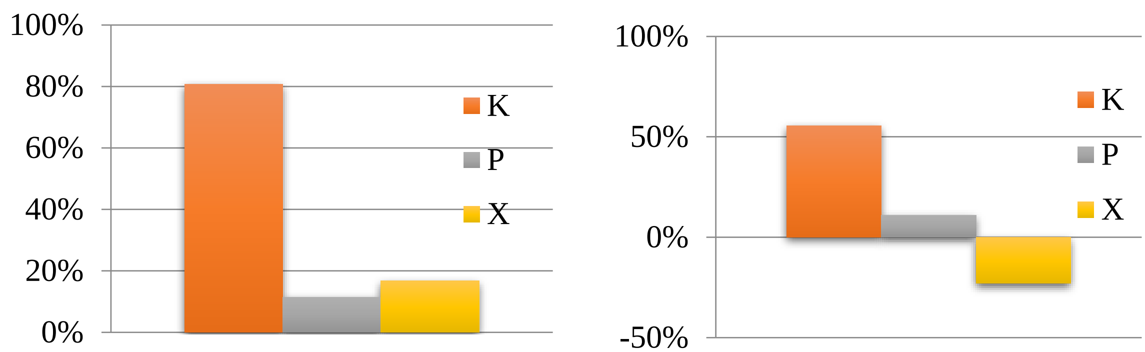
Mix	Average crack width (mm)		Maximum crack width (mm)	
	Standard conditions	Modified conditions	Standard conditions	Modified conditions
Control	0.57	0.50	1.3	1.0
K	0.11	0.22	0.6	0.62
P	0.50	0.45	1.02	0.98
X	0.47	0.62	1.0	1.2

The results in Table 2 clearly indicate that admixture K significantly reduces the average crack widths and maximum crack widths when compared to control under both standard and modified conditions. Similarly admixture P also shows some reduction in cracking when compared to control. Even though admixture X showed reduction in crack width under standard condition, it was higher than that measured for control under modified conditions. This indicates that some water proofing admixtures may be less effective at reducing shrinkage cracking under hotter and dryer conditions.

3.2. Crack reduction ratio

ASTM C1579-06 specifies that a crack reduction ratio (CRR) can be calculated using the following formula:

$$CRR = \left[1 - \frac{Average\ Crack\ Width\ of\ Modified\ Concrete}{Average\ Crack\ Width\ of\ Control\ Concrete}\right] \times 100\% \qquad (1)$$



a) Standard environmental conditions **b)** Modified environmental conditions

Figure 4. Crack reduction ratio over control

The graphs in Figure 4 compare the crack reduction ratio of each admixture when compared to control concrete mixture. It is evident that admixture K is the most effective in reducing the crack width. When admixture K was added at a dosage of 2% by mass of cement, the crack reduction ratio under standard conditions was about 80% and that under modified conditions was approximately 55% (shown in Figure 5). In comparison, the dosage for admixture X was only half of K, but this resulted in a CRR of only 15% under standard conditions and a negative CRR greater than -20% under modified conditions (meaning increasing the crack width). Admixture P at 0.8% by mass of cement had a modest CRR of about 10% under both environmental conditions.

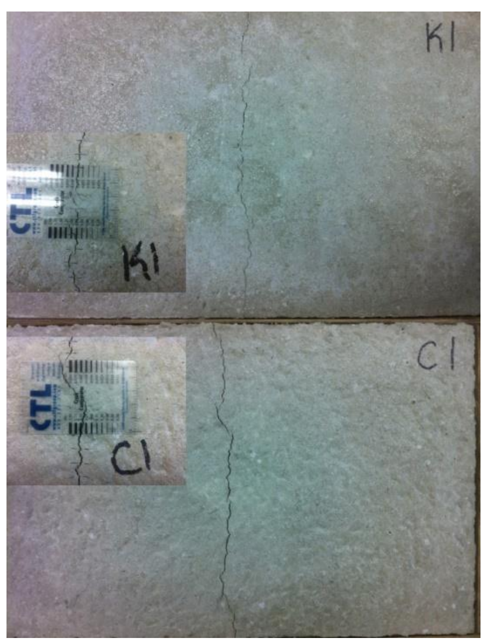
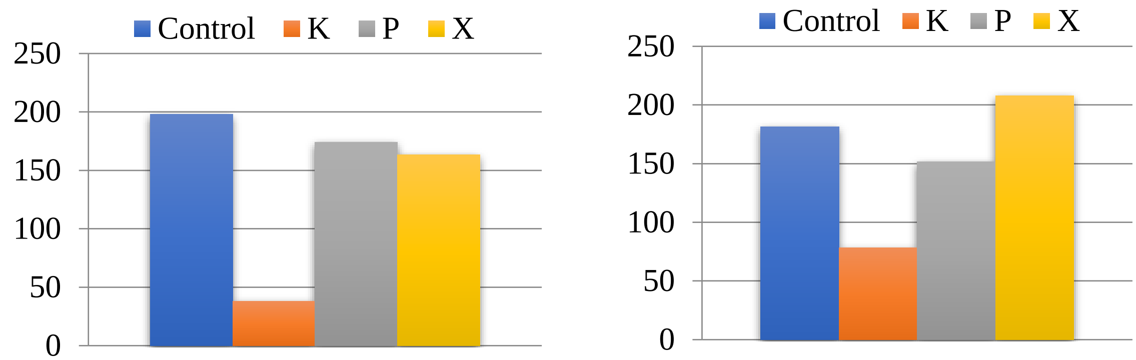


Figure 5. Crack widths of Control (C1) when compared with K1 (modified conditions)-indicating an approximate CRR of 55%.

3.3. Crack Area

Based on the crack width measurements, crack areas were calculated for all the specimens. The average crack areas for standard and modified conditions is shown in Figures 6a and 6b respectively. It is clear from the figures that the average crack area for the control was higher than 150mm2. All admixtures reduced the crack area under conditions specified by ASTM and under modified conditions. The only outlier was X that showed an increase in cracking under modified conditions. At the specified dosage, admixture K seems to be the most effective in reducing shrinkage induced cracking.



a) Standard environmental conditions **b)** Modified environmental conditions

Figure 6. Average crack area (mm^2)

3.4. Crack time and future scope of research

The following tables list out the time it took for each slab to crack. The time of cracking was observed visually from a video recording.

Table 3. Measured time to first crack.

Mix	Time to first crack (h:min)	
	Standard conditions	**Modified conditions**
Control	1:21	1:41
K	2:15	2:29
P	1:24	1:49
X	2:16	2:22

The time to first crack for the control specimens when compared to the mixes with admixtures is the lowest under both standard and modified conditions. This indicates that all the admixtures delay in formation of the first crack. Admxitures K and X delayed the time to first crack when compared to the control by an average of approximately 50mins considering both standard and modified conditions. The delay in time to first crack due to admixture P when compared to control was only an average of 5 mins.

It is also observed that the time to first crack increased under modified conditions not just for control but all specimens by an average of 16 mins. Under modified conditions, when the temperature is higher, the rate of strength gain would be faster. A delay in time to first crack under modified conditions for the control mix and the mix modified with admixture P translated into narrower cracks and slightly lower total crack area (refer to Table 2 and Fig. 6). However, for admixture K and X, larger crack areas were recorded under modified conditions. When compared to control, admixture K caused the largest delay in time to first crack and resulted in the highest CRR. Hence, no conclusive trends can be drawn from the available data and further research is warranted to confirm the relation between time to first crack and cracking. The influence of mixture proportion and additives such as fly-ash on evaporation rate has been described in a study by Banthia and Gupta (Banthia & Gupta, 2009). Future studies should focus on measuring the evaporation rate directly from the specimens as opposed to measuring the evaporation in the environmental chamber. Evaporation of moisture from the specimens is also a function of bleeding, hence measuring loss of moisture from the specimens directly can help capture the effect of bleeding as well. However, the large specimen size specified in ASTM poses challenges in accurately measuring the moisture loss from the specimens. The reduction in cracking observed in this study could be due to a faster rate of strength gain in at early-ages. To confirm this hypothesis future studies will focus on measuring the moisture loss of the specimens and also measuring the simultaneous increase in early-age strength gain of the material using dog-bone shape specimens as utilized in a study by Gupta (Gupta, 2008).

4. CONCLUSIONS

In this paper, the effects of crystalline waterproofing admixtures on restrained plastic shrinkage was examined under two environmental conditions, one specified by ASTM C1579 and the other modified. Three types of admixtures were used and added in concrete per the dosage prescribed by the manufacturers. Under both environmental conditions the samples with admixtures tended to resist cracking better than a control of the same mix proportions. Admixture K had a crack reduction ratio of approximately 80% and 55% under the standard and modified conditions respectively. The marked decrease proves that the admixture K resists plastic shrinkage effectively. The Admixture P maintained a 10% crack reduction ratio in both environmental conditions. Finally the Admixture X showed a small decrease in crack size in the standard condition. However, it performed poorly in the modified condition with it cracking more than the control. Overall, commercially available crystalline water proofing admixtures seem to offer the secondary benefit of serving as a shrinkage reducing admixture especially at early age.

5. ACKNOWLEDGEMENTS

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Damage assessment in concrete structures using piezoelectric based sensors

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ABSTRACT
Piezoelectric based PZT (Lead Zirconate Titanate) smart sensors offer significant potential for continuously monitoring the development and progression of internal damage in concrete structures. Changes in the resonant behavior in the measured electrical conductance obtained from electro-mechanical (EM) response of a PZT bonded to a concrete substrate is investigated for increasing levels of damage. Changes in the conductance resonant signature from EM conductance measurements are detected before visible signs of cracking. The root mean square deviation of the conductance signature at resonant peaks is shown to accurately reflect the level of damage in the substrate. The findings presented here provide a basis for developing a sensing methodology using PZT patches for continuous monitoring of concrete structures.
Keywords: PZT; electro-mechanical impedance; conductance; microcracks.

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Evaluación de daños en estructuras de concreto utilizando sensores piezoeléctricos

RESUMEN

Los sensores inteligentes PZT (Lead Zirconate Titanate) basados en piezoeléctricos ofrecen un potencial significativo para monitorear continuamente el desarrollo y la progresión de los daños internos en estructuras de concreto. Se investigan los cambios en el comportamiento resonante a través de la conductancia eléctrica medida, obtenida a partir de la respuesta electromecánica (EM) de un PZT unido a un sustrato de concreto para aumentar los niveles de daño. Los cambios en la resonancia de la conductancia EM se detectan antes de que aparezcan signos visibles de agrietamiento. La desviación cuadrática media de la raíz de la conductancia en los picos resonantes refleja con precisión el nivel de daño en el sustrato. Los hallazgos presentados aquí proporcionan una base para desarrollar una metodología de detección utilizando parches PZT para el monitoreo continuo de estructuras de concreto.

Palabras clave: PZT; impedancia electromecánica; conductancia; microfisuras.

Avaliação de danos em estruturas de concreto usando sensores piezoelétricos

RESUMO

Os sensores piezoelétricos inteligentes PZT (Lead Zirconate Titanate) oferecem um potencial significativo para o monitoramento contínuo do desenvolvimento e progressão de danos internos em estruturas de concreto. As alterações de ressonância através da medida da condutância elétrica obtida a partir da resposta eletromecânica (EM) de um PZT ligado a um substrato de concreto é investigada para níveis crescentes de danos.

As alterações no perfil de ressonância de condutância EM são detectadas antes de sinais visíveis de fissuras. O desvio quadrático médio da raiz do perfil de condutância nos picos ressonantes é mostrado para refletir com precisão o nível de dano no substrato. Os resultados aqui apresentados fornecem uma base para o desenvolvimento de uma metodologia de detecção usando PZT para monitoramento contínuo de estruturas de concreto

Palavras chave: PZT; impedância eletromecânica; condutância; microfissuras.

1. INTRODUCTION

Structural Health Monitoring (SHM) is a process of assessing the structural integrity of the constituent parts and the level of damage in the structure during its life period. SHM relies on non-destructive evaluation (NDE) procedures and continuous monitoring of structural parameters to determine the intensity and location of the damage. This involves sensors, data acquisition system and signal processing tools. Signs of distress in concrete are often associated with visible cracking. Since concrete is a brittle material, which is weak in tension, cracking is the manifestation of damage in the material which results from tensile stress in the material. Stress induced damage in concrete could result from load application or from internal sources such as shrinkage or corrosion of reinforcing steel. Damage initiation takes place in the form of distributed microcracks, which eventually localize to form cracks. Often the damage, particularly in the incipient stages is not directly visible and by the time signs of distress appear on the surface of the structure, significant damage would have accrued in the structure and there may be significant degradation of the capacity of the structure. Early detection of damage, before visible signs appear on the surface of the structure is essential to initiate early intervention, which can effectively increase the service life of structures. Methods to detect incipient damage in the form of microcracks are required to provide effective methods of monitoring structural health and service life performance of structures.

Use of PZT patches and wafers has become popular in structural health monitoring. Due to the coupled electro-mechanical constitutive response of a PZT material, the mechanical response of a bonded PZT patch subjected to an applied electrical potential is influenced by the elastic restraint provided by the substrate material. Coupling the PST patch to a structure changes the mechanical impedance of the PZT, which produces changes in its vibration characteristics. Monitoring changes in the electrical impedance signature due to changes in the effective mechanical impedance of the substrate is the basis for electromechanical (EM) impedance-based measurements. Information about the surrounding material is contained in the electromechanical impedance (EMI) signature of a PZT. By comparing the impedance signature taken in the pristine state and at any other time, structural damage can be determined. Generally, both frequency and amplitude shifts are produced relative to the pristine state (without damage) (Ayres et al., 1998; Chaudhry et al., 1995; Sun et al., 1995; Park et al., 2000; Zagrai and Giurgiutiu, 2001; Giurgiutiu et al., 2002, 2004; Peairs et al., 2004; Narayanan and Subramaniam, 2016a).

Application of EMI technique for damage detection in concrete structures requires a careful study of the changing compliance of the substrate for different forms of damage in the substrate material from the incipient to the visible stages. The use of PZTs for health monitoring of concrete structure was demonstrated by the ability of EMI technique to register changes due to formation of cracks well in advance of failure (Park et al., 2000; Narayanan and Subramaniam, 2016b). Several other studies of damage in concrete using impedance-based measurements of PZTs have been conducted using embedded defects and artificial damage in the form of machine cuts (Tseng and Wang, 2004; Lim et al., 2006; Dongyu et al.,2010; Wang et al., 2013). The EM impedance method has also been used to determine the location of a crack by inducing crack at different positions and depths and cross correlation as damage index (Wang et al., 2013). While the use of artificial damage provides meaningful insight, it is not representative of substrate compliance with stress/load induced damage in the material.

The potential use of EMI based measurements of surface mounted PZT to identify the formation of incipient damage in concrete structures, is evaluated in the paper. The relationships between forms of material damage, visual indication of damage, mechanical compliance of the material and resonant modes in the conductance signature of PZT bonded to a concrete substrate are investigated. The variation in surface strains for incremental levels of loading is monitored using Digital Image Correlation (DIC) and compared with the conductance plot of the PZT. Root mean square deviation (RMSD) of the EM conductance close to the resonant peak is used as a damage index and variation in RMSD at different damage states is presented.

2. EXPERIMENTAL PROGRAM

Experiments were performed using 150 mm concrete cubes. Six cubes were cast and cured for 90 days before testing. The cubes were bonded with PZT patches exactly at the center of the side face of the cube using a two-component epoxy. The properties of the concrete and epoxy are given in Table 1. Three cubes were tested to failure to determine the compressive strength of the concrete.

Table 1. Material properties

Type	Average Failure stress (*MPa*)	Young's Modulus (*GPa*)	Density (ρ) (kg/m^3)	Poison's ratio(υ)
Concrete cube	52	36	2300	0.2
Epoxy	-	2	1400	0.36

The front faces of the cubes were smoothened and a sprayed-on speckle pattern was created for measurement of surface displacements using the full-field optical technique known as digital image correlation (shown in Figure 1a). The baseline signatures of the PZT when attached to the substrate were taken. 20mm x 20mm PZT patches of 1 mm in thickness were used in the experimental study. In a typical impedance measurement, the frequency was varied between 1 kHz and 0.5 MHz at an applied voltage of 1V and data was collected at 800 discrete frequencies. Average of five measurements was collected. Impedance data was collected from the PZT patch in the free-state before attaching the PZT to the concrete cube. The baseline EM conductance signature and image were taken prior to the start of loading. Cubes were subjected to cyclic compressive loading of increasing magnitude where the load amplitude was increased in increments of 10% of the average compressive strength in every cycle. The loading procedure consisted of alternate loading and unloading cycles as shown in Figure 1b. During the loading, the conductance signatures and the image for DIC were recorded on top of the load cycle and after unloading.

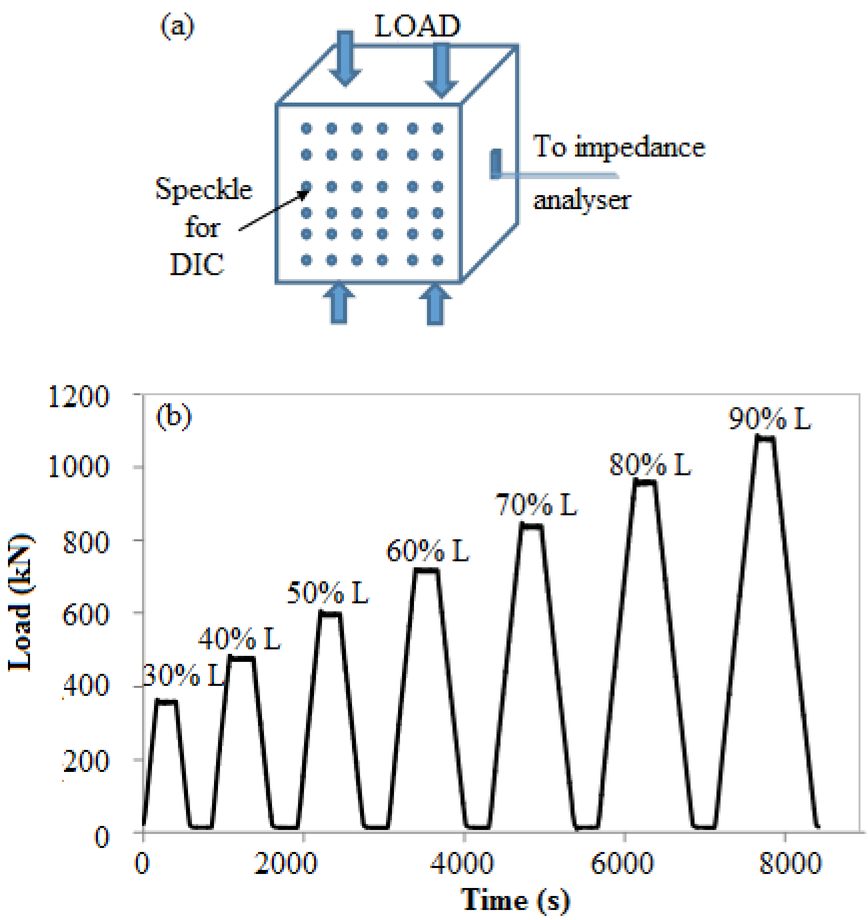

Figure 1. (a). Experimental set-up (b). Applied loading history

3. ELECTRO-MECHANICAL IMPEDANCE OF PZT

In a PZT material, the application of an electrical field results in mechanical strain in the material due to the coupled electro-mechanical constitutive relations. For a PZT patch attached to a substrate subjected to an applied electrical input, the motion of the interface subjected to continuity conditions is governed by the combined mechanical impedance of the structure and the PZT. The constrained motion in turn produces a change in the measured electrical impedance. The first systematic attempt to derive the electrical impedance of the PZT which is mechanically connected to a structure using a 1D idealization of the system was developed by Liang et al., 1994. Subsequent improvements in modelling the PZT response have included the effective 1-D model of the PZT and varying levels of idealization of the structural impedance (Bhalla et al., 2004; Xu and Liu, 2002; Yang et al., 2005,2008). Most of the available analytical solutions are applicable for 1 or 2-D idealizations of the PZT, substrate or both. Typically, the complex electrical admittance ($\bar{Y}$) of the PZT patch for a given electrical input at a frequency can be represented as a function of $\bar{Y}(Z_A, Z_S, \omega, l_i, E)$.where Z_A And Z_S are the mechanical impedance of the PZT and substrate respectively. l_i, represent the dimensions of patch (length, breadth or thickness) and E is the electric field applied for actuation with circular frequency ω .
The conductance, which is the real part of admittance of the free PZT and the PZT bonded to the 150 mm concrete cube are shown in Figure 2. It can be seen that resonance peaks associated with the free vibration of the PZT can also be identified in the response of the PZT attached to the concrete cube. Only three prominent peaks are identified in the conductance spectrum of the bonded PZT. Peaks 1 and 2 in the conductance spectrum of the bonded PZT correspond with modes 1 and 3 respectively of the PZT. The third peak in the conductance response of the bonded PZT has contributions from closely spaced modes 5 and 6 of the PZT. There are several prominent changes associated with the frequency of the resonant modes and the relative magnitude of the resonant peaks. There is a noticeable decrease in values of conductance, an increasing baseline trend which increases the magnitude of conductance with increasing frequency and a change in the relative magnitudes of the resonant peaks in the bonded state. There is also a significant broadening of the resonance peaks compared with the free-state.
The resonance peaks shift to higher frequencies, with a larger frequency shift in lower modes. The resistance to the motion of the PZT by the substrate is reflected in the overall decrease in the value of conductance. While the conductance of the free PZT is essentially zero between resonant peaks, the conductance is non-zero between the resonant peaks for the bonded PZT. The resistance to the motion of a point located on the surface of the cube, given by the driving point impedance, influences the motion of the bonded PZT. The frequency dependency of the substrate driving point impedance is reflected in the relative shifts in the amplitudes and the general increasing trend in the background of the measured conductance. The influence of the substrate can also be identified with the overall increase in the frequency and broadening of the resonant peaks.

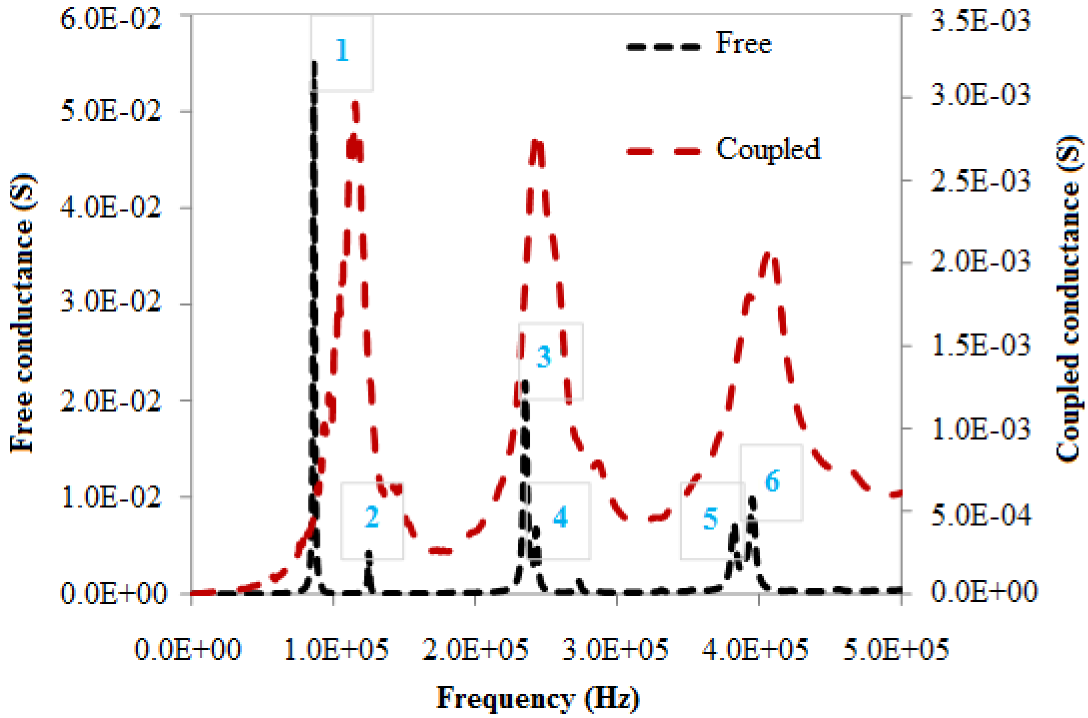

Figure 2. Conductance spectrum of PZT in the free condition and coupled with a 150 mm concrete cube.

4. ANALYSIS OF RESULTS

From the results of the numerical analysis of surface bonded PZT carried out in COMSOL multiphysics^{TM}, the first peak was not well defined for concrete. The second peak was well defined and sensitive to the change in elastic modulus. The second peak in the EM conductance response of the bonded PZT was selected for evaluating the influence of load-induced damage. The conductance signatures at the second peak of the bonded PZT response after unloading from different load levels are shown in Figure 3 a, b.
The second peak is centered on 255 kHz. The response between 245 and 265 kHz is plotted in the figures. Contours of horizontal strain at distinct loading obtained from the DIC technique are shown in Figure 4. It can be clearly identified from the plot that the unloading signature at 40%u shows a shift to lower frequencies. This is due to the incipient damage produced in the concrete. Horizontal strain contour shows an increase in strain levels (Figure 4). As the load level increases, the resonance peak in the conductance signature shows a consistent leftward shift. Comparing with the measured DIC response, there is no visible sign of distress or cracking up to 70% of strength, while some signs of localization are evident at 60% of peak. Localization of damage into a crack occurs at 70% of strength. Significant changes in the resonant peak associated with the localization are observed. After localization, significant changes are observed in the shape of the resonant peak. At 90% of the compressive strength, the peak showed a significant decrease in amplitude and a flattening of the peak. The flattening of the peak is associated with the formation of a major crack on the surface. The conductance signatures associated with the resonant peak has a very good agreement with the indication of damage obtained from surface strain measurements. Further, changes in EM conductance are observed before any visible sign of distress.

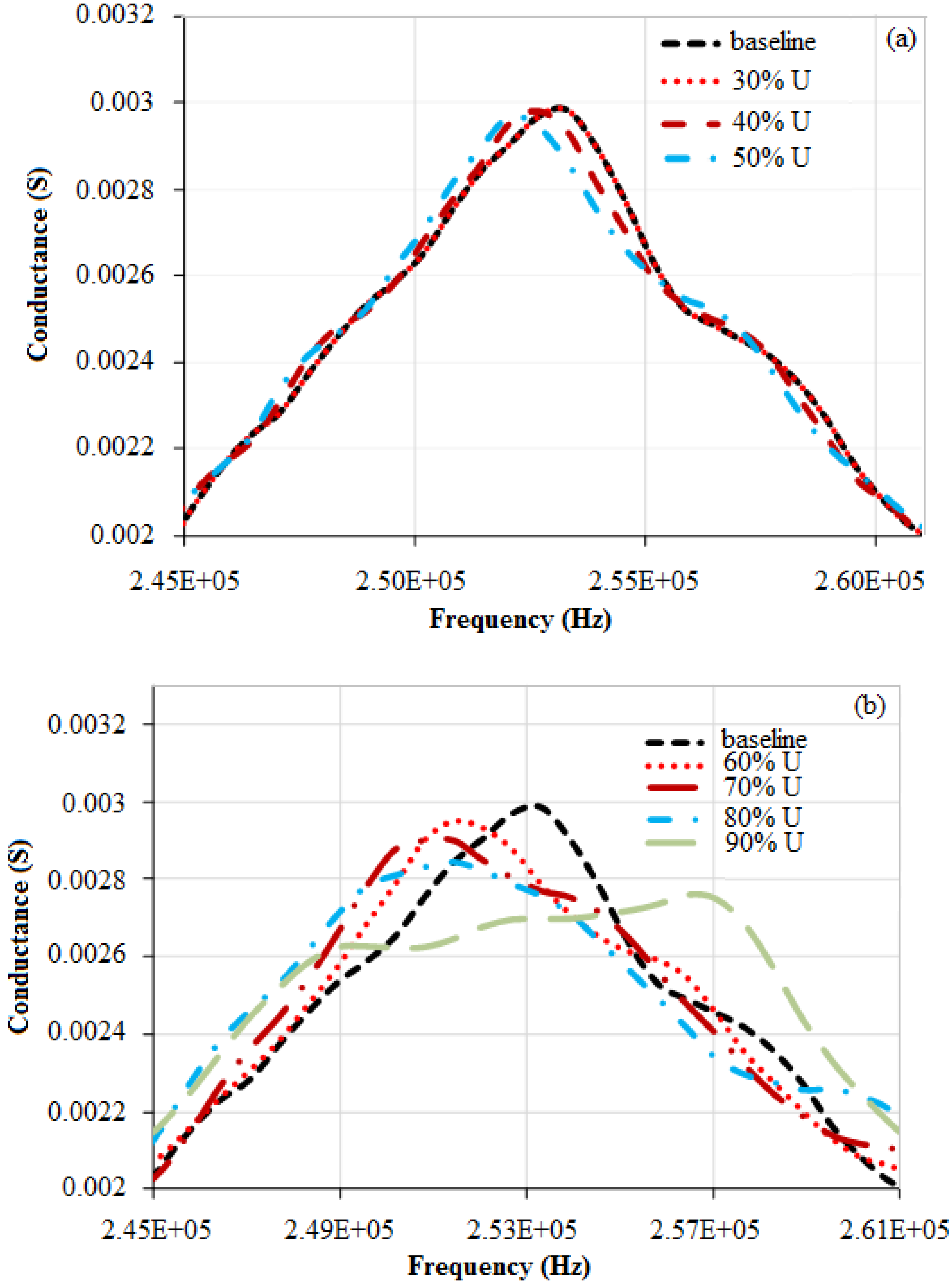

Figure 3. Electrical conductance signatures: a. 30%-50% of strength b. 60%-90% of strength

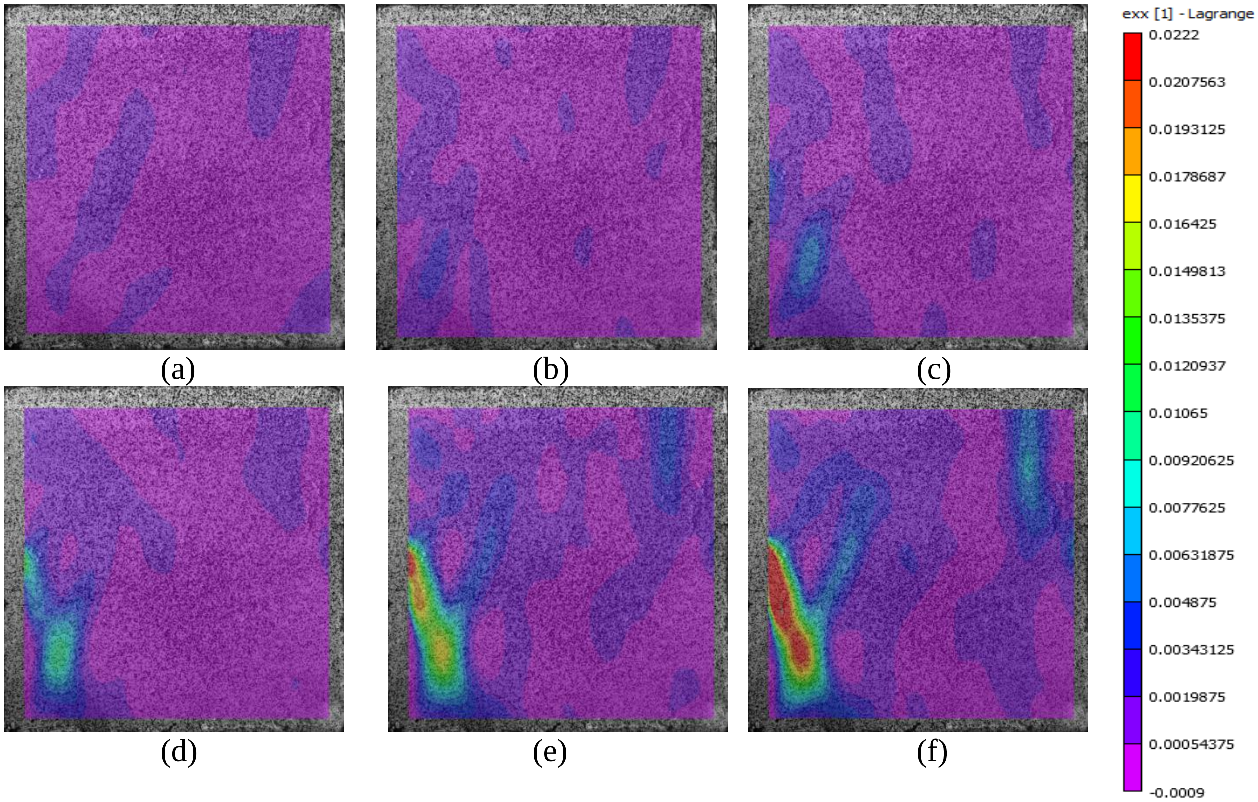

Figure 4. Contours of horizontal strain (e_{xx}) obtained using digital image correlation (a) at 40%; (b) at 50%; (c) at 60%; (d) at 70%; (e) at 80%; and (f) at 90% of Strength.

The root-mean-square deviation (RMSD) is used to measure the differences between values of baseline measurement of conductance signature at the second resonant peak and the corresponding signatures at different load levels. The RMSD for the frequency range 245 kHz-260 kHz with respect to the baseline measurement were calculated using equation (1), where. x_i and y_i are the signatures obtained from the PZT transducer bonded to the structure before and after damage (or loading) with length N. The scatter in the results obtained from all the specimens is also plotted in the figure.

It can be seen that despite the scatter, there is an increasing trend of RMSD with each level of loading as shown in the Figure 5a. The variation in the average vertical strains recorded at the top and bottom of the load cycles obtained from DIC measurements are also plotted in Figure 5b. It can be seen that the level of damage assessed using the RMSD variation of the second resonant peak compares well with the evolution of plastic strain and increase in mechanical compliance. There is an exponential increase in the evolution of plastic strain with loading. Plastic strain is an indicator of level of damage in the material. This corresponds with the observed trend in the RMSD measured with loading.

$$RMSD = \sqrt{\frac{\sum_{i=1}^{N}(y_i - x_i)^2}{\sum_{i=1}^{N} {x_i}^2}} \qquad (1)$$

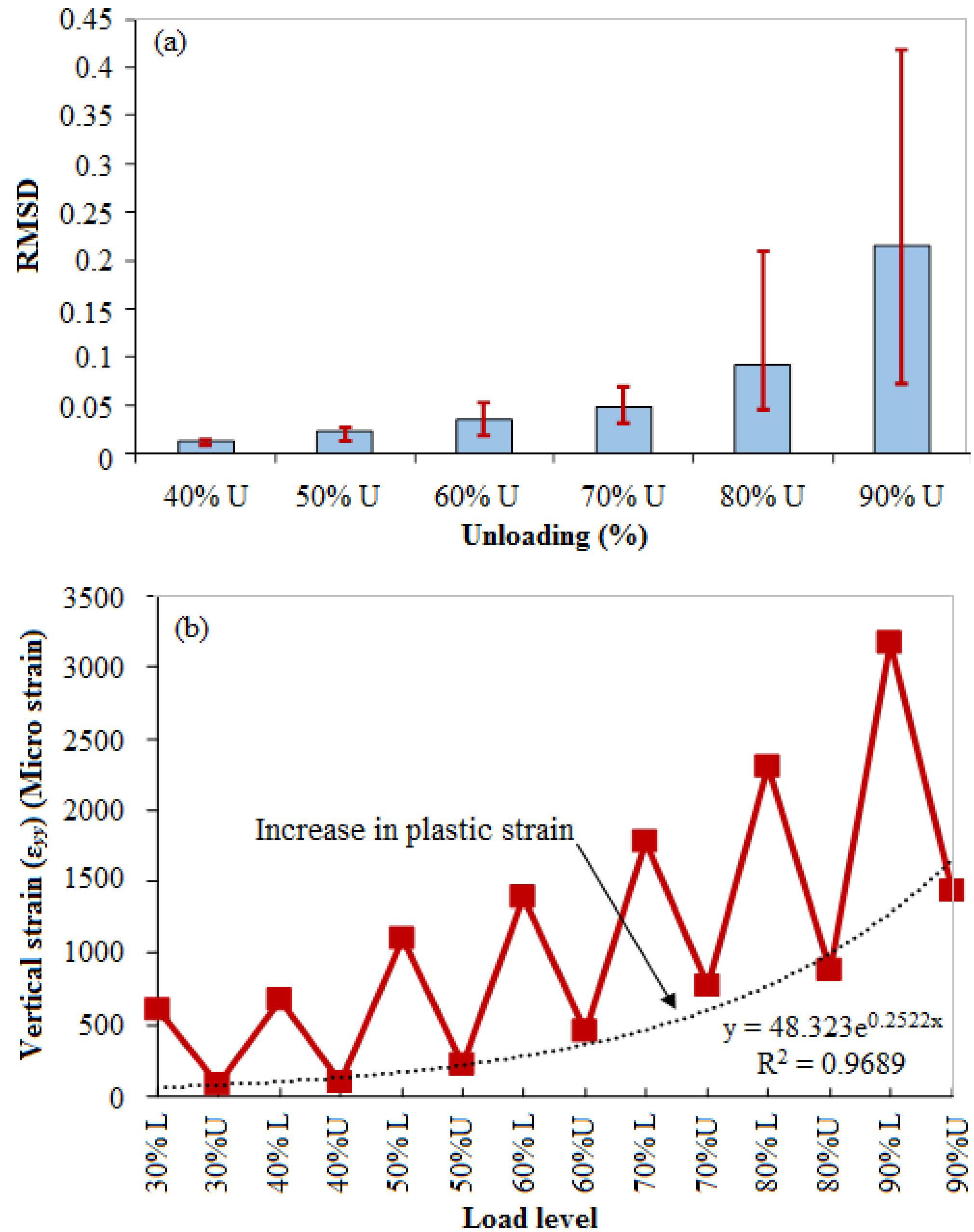

Figure 5. (a). RMSD of the second resonance peak (b). Average vertical strain (ε_{yy}) obtained from DIC

5. CONCLUSIONS

The potential of using EM impedance measurements of surface mounted PZT patches for structural health monitoring of concrete structures is established. It is shown that there are changes in resonant behavior of the EM conductance response of the PZT bonded to a concrete substrate with increasing damage. The PZT sensor detects incipient damage significantly earlier than the appearance of visible signs of damage. There is an amplitude reduction and frequency shift of the PZT resonance peak with an increase in damage in the concrete substrate. At higher damage levels, there is flattening of the resonant peak associated with localization and formation of a major crack.

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Proposal for improved mixes to produce concrete masonry units with commonly used aggregates available in the Valley of Mexico

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ABSTRACT

In this paper, a proposal is done to update the masonry index compressive strength design value f^*_m for solid concrete masonry units for the masonry guidelines of Mexico´s Federal District Code (NTCM-2004). Solid units were made by taking into account the characteristics of the most commonly used raw materials available in the Valley of Mexico to fabricate such units in the Metropolitan Area of Mexico City. Different tests were conducted for both raw materials and the obtained concrete units. Based upon test results, it is illustrated why it is much better to design masonry structures based upon the experimental data of the units to be used at the construction site rather than using index values proposed in building codes.
Keywords: masonry; concrete masonry units; compressive strength.

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Propuesta de mejora de mezclas para producir piezas de mampostería de concreto empleando materiales comúnmente disponibles en el Valle de México

RESUMEN

Se presenta un estudio donde se hace una propuesta para actualizar los valores índices de la resistencia a la compresión de mamposterías (f^*_m) elaboradas con piezas de concreto especificados en las normas de mampostería vigentes en el Distrito Federal. Se realizó un estudio de mercado, donde se tomaron en cuenta las características de los materiales comúnmente utilizados en la actualidad en el Valle de México para la fabricación de piezas de concreto sólidas, incluyendo los resultados de distintas pruebas de laboratorio realizadas a la materia prima. Con base en estos resultados, se ilustran las ventajas de diseñar a la mampostería con base experimental en lugar de utilizar los valores indicativos que se ofrecen en las normas de mampostería.

Palabras clave: mampostería; piezas de concreto; resistencia a la compresión.

Proposta de melhoria de traços para produzir blocos de alvenaria de concreto utilizando materiais comumente disponíveis no Vale do México

RESUMO

Apresenta-se um estudo onde se propõe uma atualização dos valores do índice de resistência à compressão da alvenaria (f^*_m) elaborados com blocos de concreto especificados nas normas de alvenaria em vigor no Distrito Federal. Foi realizado um estudo de mercado, que levou em conta as características dos materiais comumente utilizados atualmente no Vale do México para a fabricação de blocos de concreto sólidos, incluindo os resultados de vários testes de laboratório nas matérias-primas disponíveis. Com base nestes resultados, são apresentadas as vantagens de projetar a alvenaria com base experimental em lugar de usar os valores indicativos oferecidos nas normas de alvenaria.

Palavras chave: Alvenaria; blocos de concreto; resistência à compressão.

1. INTRODUCTION

In Mexico, most housing construction is built with masonry. In most housing applications, main structural elements are composed of masonry to resist gravitational and lateral loads. Structural masonry infill walls are also used. Masonry is used in other structural elements such as fences and buttresses. The non-structural use of masonry as partition walls is widespread, as architects and users appreciate their property to reflect sound waves, then providing excellent sound isolation between adjacent rooms for the comfort of building users, a property that cannot be offered with other lighter options such as plywood walls, for example. Confined masonry is the dominant mode of construction in engineered projects and self-construction projects in cities, whereas unreinforced masonry is used in non-engineered construction, primarily in small towns and the countryside. Reinforced masonry is rarely used within the country. Given its extended use as structural material in Mexico, it is very important to build using quality masonry units with adequate strength and durability properties to resist the actions to which they will be subjected.
Unfortunately, the quality of most masonry units available in Mexico City and its suburban area has decayed in past decades, particularly the concrete masonry units. When the masonry guidelines of Mexico´s Federal District Code were published by the first time (NTCM-77, 1977), tables were provided for the compressive design strength of the masonry (f^*_m) in terms of the

compressive strength of the masonry units (f^*_p) and the mortar used based upon the experimental program developed during the 1960s-70s at the Institute of Engineering of the National Autonomous University of Mexico (UNAM, acronym in Spanish), as reported in Meli (1979) and partially reproduced in Tena and Miranda (2002).

A histogram of the concrete masonry units produced in Mexico City at the times was reported by Meli (1979) and it is shown in Figure 1. At the times, there were three kinds of concrete masonry units produced in Mexico City: a) heavy weight, b) normal weight and, c) light weight. Higher strengths and less scatter were obtained for the heavy concrete masonry units because they were manufactured in industrialized factories using aggregates with few porosities and high cement/aggregate ratios. Lower strengths and higher scatter were obtained for light weight masonry units, because they were not necessarily manufactured in factories with high quality-control standards and they used aggregates with high porosities and small cement/aggregate ratios. In fact, it was observed that light weight concrete blocks were prone to be damaged during the transportation loading and unloading process, producing more waste or having the risk that damage units would be actually used in masonry elements (Tena and Miranda, 2002).

It can be observed from the histogram depicted in Figure 1 that in the 1970s, the quality of the concrete masonry units produced in Mexico City Metropolitan Area was adequate for structural use, as the mean compressive strength for the units was $\overline{f}_p = 115$ kg/cm^2. It can also be observed from Figure 1 that most of the tested concrete masonry units had a compressive strength in the range of 80 to 120 kg/cm^2.

However, it was found since then that the solid concrete building brick exhibited a high strength scatter among manufacturing factories because they used different aggregates and the cement/aggregate mixes varied. For the solid concrete building brick, the obtained mean compressive strength was $\overline{f}_p = 57$ kg/cm^2, with a high coefficient of variation of 0.54 or 54% (Meli, 1979; Tena and Miranda, 2002). Besides its low compressive strength, the solid concrete building brick exhibits a high volumetric expansion and a high permeability, therefore, making its use unattractive for applications with frequent contact with the water or in environments with high humidity (Meli, 1979; Tena and Miranda, 2002).

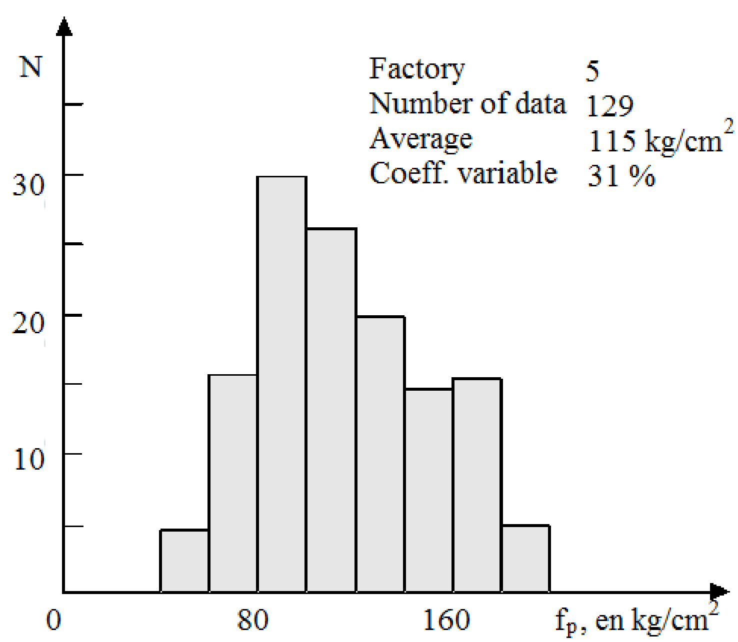


Figure 1. Histogram for the compressive strength of concrete masonry units (Meli, 1979)

Regrettably, this solid concrete building block is the cheapest manufactured masonry material and therefore, it is massively used by low-income people for housing purposes in Mexico, Mexico City included. It is even more unfortunate that its quality has lowered with time. In fact, not only the quality of the solid concrete building block has lowered in Mexico City, but also for the concrete blocks, where a great porosity is easily observable and even one can easily break them with the hand at the time of grabbing them!

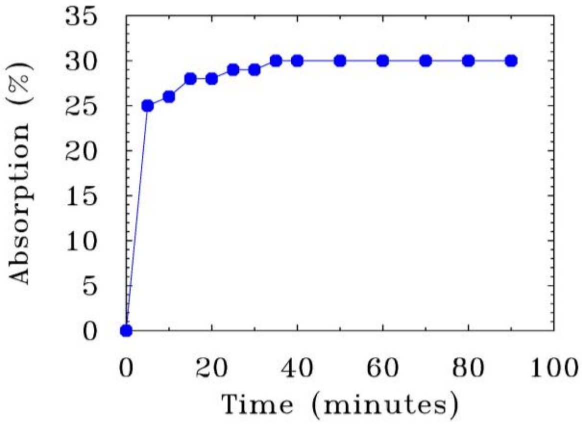
It is worth noting that in recent testings conducted at the Metropolitan Autonomous University, Azcapotzalco campus (UAM-A, acronym in Spanish), the most common concrete blocks that are sold and used to build combined and confined masonry structures had even lower quality. From the testing of 18 concrete blocks produced by the same manufacturer, the obtained average compressive strength was $\bar{f}_p = 43.3$ kg/cm^2 and the average water absorption was 26.5% (Tena *et al.*, 2007; Tena-Colunga *et al.*, 2009).

Therefore, this research team started an experimental program intended to find practical solutions to solve the already ancient problem of having mostly low quality concrete blocks (solid and hollow) available in the Valley of Mexico. The program started finding practical solutions to improve their main properties while using the same raw materials (aggregates) currently used by the manufacturers that supply the building market of the Metropolitan Area of Mexico City. Some aspects of the conducted study are presented in following sections and the details are reported elsewhere (Liga and Pérez, 2013).

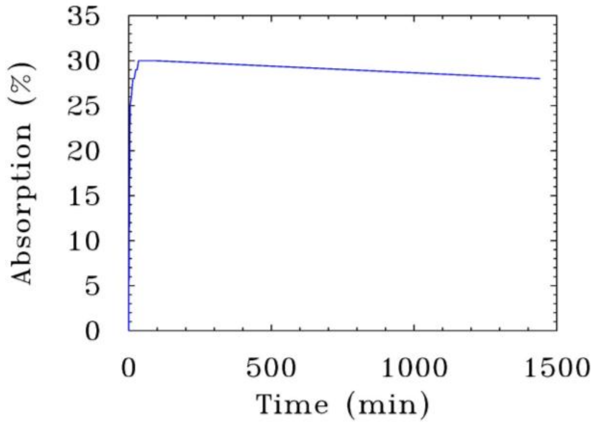
2. CONCRETE UNITS PRODUCED IN THE VALLEY OF MEXICO

The first step of the conducted research was to quickly assess the properties of most common solid concrete building blocks currently marketed in the Valley of Mexico. The concrete units were supplied by a manufacturer that uses "tepojal" (a material with fineness modulus of 4.16) as its main aggregate, given its abundance in the valleys of Mexico and Toluca. The manufacturer uses a Portland cement classification CPC 40, produced by Lafarge. Nominal dimensions for the solid concrete building block units are 7 cm x 12.5 cm x 25 cm. These units are commercialized in several construction retail stores.

Water absorption and compressive strength tests were conducted. Six units were used for the water absorption tests. Measurements were done at small time intervals (5 and 10 minutes), as well as a time of 1.5 and 24 hours. The average water absorption curves obtained for the time intervals under study are shown in Figure 2, where it can be observed that the water absorption rate is very high, surpassing for 24 hours (Fig. 2b) the 20% limit established in the Mexican norm NMX-C-404 (2005). In fact, the units absorb more than 25% their weight in only 5 minutes (Fig. 2a).



a) Absorption in 1.5 hours



b) Absorption in 24 hours

Figure 2. Water absorption for some concrete units marketed in the Valley of Mexico

Compression tests were conducted according to what it is established in Mexican masonry guidelines NTCM-04 (2004) and Mexican norm NMX-C-036 (2004). Therefore, nine units were tested, using neoprene plates ½'' thick as pitching material (Fig. 3a). It was observed from the tests the typical lateral split tension failure for the units (Fig. 3b). The obtained average compressive strength was $\overline{f}_p = 25.5$ kg/cm^2, which it is considerably lower than the requested minimum strength for the masonry units in norm NMX-C-404 (2005) and in the masonry design guidelines NTCM-04 (2004), where the minimum required design strength is f^*_p=60 kg/cm^2. In fact, according to NTCM-2004, the required minimum average compressive strength for structural use is $\overline{f}_p = 105$ kg/cm^2, taking into account that, at the time of assessing f^*_p: a) the minimum coefficient of variation established in NTCM-04 for the tested units is 0.30, and, b) a statistical criterion corresponding to the 98 percentile is considered in NTCM-04 (Alcocer *et al.*, 2003).

Then, with the obtained test results it was confirmed the suspicion that this research team had taking into account previously available experimental information: it is alarming the low quality of many concrete masonry units that are produced and marketed in the Valley of Mexico and that are sold in many construction retail stores, despite the fact a quality cement was used in their fabrication. Therefore, it was decided to manufacture solid concrete building block units of the same dimensions and using the same raw materials (aggregates), but different mixes. The purpose was to obtain units with reasonable strengths for structural use according to what it is established in NMX-C-404 (2005) and NTCM-04 (2004).

a) Unit in testing machine

b) Typical unit failure

Figure 3. Compression tests of some solid concrete units marketed in the Valley of Mexico

3. DESIGN OF MIXES TO SATISFY MEXICAN NORMS

Given the low quality of many of the concrete masonry units that are sold and used in the Valley of Mexico, it was decided to design concrete mixes using the same aggregates and cement currently used by the manufacturer, but trying to improve their average compressive strength. In order to have a reasonable strength range, mixes were designed to obtain an average compressive strength $\overline{f}_p$ between 50 kg/cm^2 and 200 kg/cm^2. The main reason to target such a strength range was to build later masonry prisms using such units and the mortars allowed in NTCM-04. Then, conducting compressive tests to assess f^*_m, the obtained results can be used to propose updated tables for NTCM for the design compressive strength of the masonry (f^*_m) in terms of the compressive strength of the concrete masonry units (f^*_p) and the mortar.

From the field investigation (Liga and Pérez 2013) it was obtained that most manufacturers use tepojal as their main aggregate to produce solid concrete building block units and, depending on the zone and material availability, other manufacturers use tezontle and sand as their base aggregates.

It is known as tepojal (Fig. 4a) a clayly-sand which it is plenty available in Mexico. It is a small volcanic grain covered with clay, very light and with a high porosity, which in theory makes it an ideal material to produce light weight concrete masonry units. The tepojal used in this study was obtained from material banks at Toluca, and its detailed material characterization (granulometric curves and main material properties) was done according to Mexican norms NMX-C-073 (2004), NMX-C-077 (1997), NMX-C-111 (2004) and NMX-C-165 (2004), as reported in detail in Liga and Pérez (2013). The main obtained properties for tepojal were: a) fineness module: 4.16, b) water absorption: 54.25%, c) humidity: 39.26%, d) loose volumetric weight: 0.68 ton/m^3, e) compact volumetric weight: 0.81 ton/m^3, f) dried unit weight: 1.04 ton/m^3 and, g) saturated unit weight: 1.60 ton/m^3. From the results obtained, it is concluded that the tepojal used in this study is a very light material but with a very high water absorption potential; this last characteristic is not desirable for the manufacture of light weight concrete masonry units.

a) Tepojal b) Tezontle c) Sand

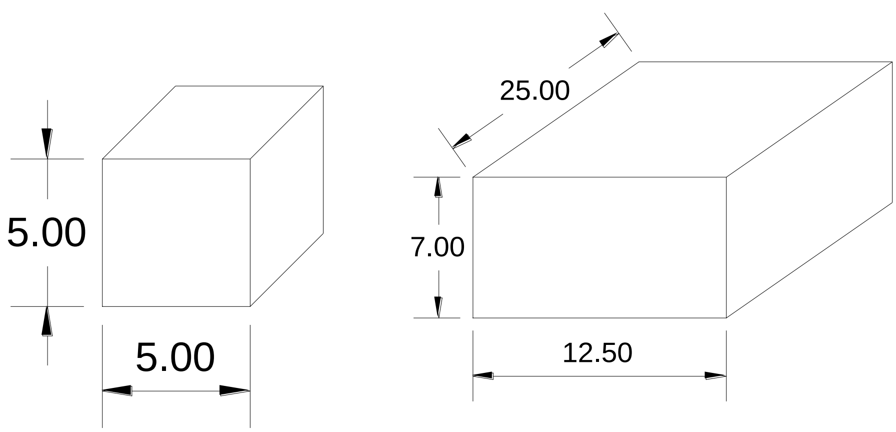
Figure 4. Used aggregates for the design of concrete mixes

Tezontle (Fig. 4b) is a red, thin gravel from volcanic origin, which it can be found in hillsides, volcanoes or depressions. The tezontle used in this study was obtained from material banks at Santa María Chiconautla in the State of Mexico. Its material characterization was done as previously described for tepojal. The main obtained properties for tezontle were (Liga and Pérez, 2013): a) water absorption: 20.46%, b) humidity: 7.78%, d) loose volumetric weight: 0.91 ton/m^3, e) compact volumetric weight: 1.04 ton/m^3, f) dried unit weight: 1.56 ton/m^3 and, g) saturated unit weight: 1.87 ton/m^3. From the results obtained, it is concluded that the tezontle used in this study is a light material with reasonable water absorption potential; so tezontle is an ideal material to manufacture light weight concrete masonry units.

Finally, common sand (Fig. 4c) from a material bank in Huixquilucan, State of Mexico, was also used. The main obtained properties from the material characterization were: a) water absorption: 28.11%, b) humidity: 1.66%, d) loose volumetric weight: 1.27 ton/m^3, e) compact volumetric weight: 1.43 ton/m^3, f) dried unit weight: 1.66 ton/m^3 and, g) saturated unit weight: 2.17 ton/m^3. From the results obtained, it is concluded that the sand used in this study is a normal weight material with reasonable water absorption properties; so, it is an ideal material to manufacture quality concrete masonry units.
Given the good properties obtained for tezontle, its granulometric curve and its cost, it was decided to use a tezontle-sand mix in a 30-70 volumetric proportion (30% tezontle and 70% sand), which it was also characterized. The main obtained properties for this material mix were: a) water absorption: 21.51%, b) dried unit weight: 1.68 ton/m^3 and, c) saturated unit weight: 2.04 ton/m^3.

3.1 Design of concrete mixes to manufacture concrete masonry units

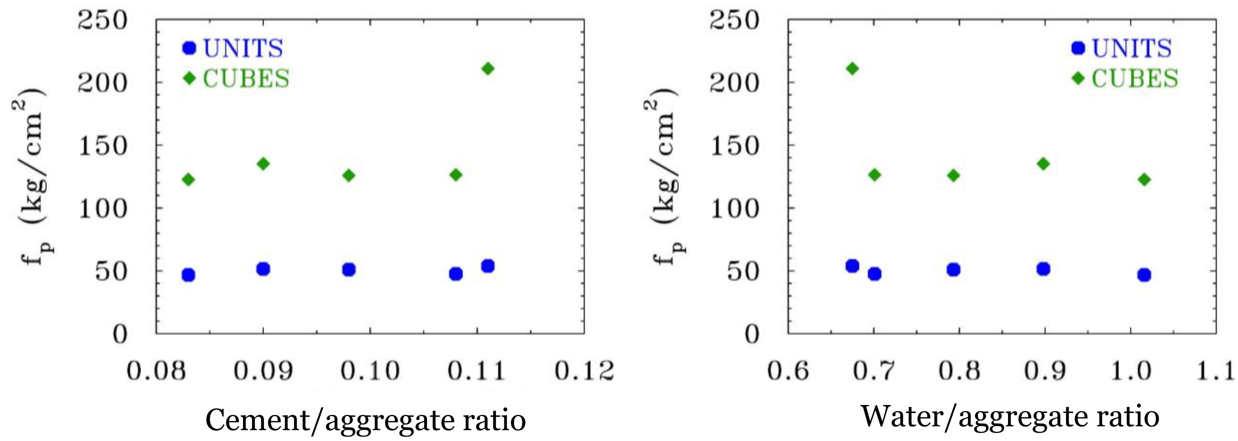
The design of concrete mixes was done following traditional methods available in the literature (for example, Neville, 1998). Different mixes were made using tepojal and the mix tezontle-sand (30-70) as base material for different cement/aggregate and water/cement ratios. Solid concrete masonry units and standard cubes were built with the resulting mixes (Fig. 5). The initial objective was to obtain average compressive strengths close to 50, 100, 150 and 200 kg/cm^2. The details of the concrete mix design and proportioning and their related experimental results are reported in greater detail in Liga and Pérez (2013).
The project started with the design of concrete mixes using tepojal as base material. Compressive strength tests for the manufactured solid units and standard cubes (Fig. 5) were conducted. The obtained initial results from the compressive strength test were discouraging. A big difference was observed for the individual and average (Fig. 6) compressive strength between the cubes and the solid units for the same concrete mix. It is clear then that the shape factor affects the obtained results, but no information was available on how to relate cube and units strengths (strength correction factor).



a) cubes   b) solid masonry units

Figure 5. Dimensions (cm) of cubes and masonry units tested under axial compression

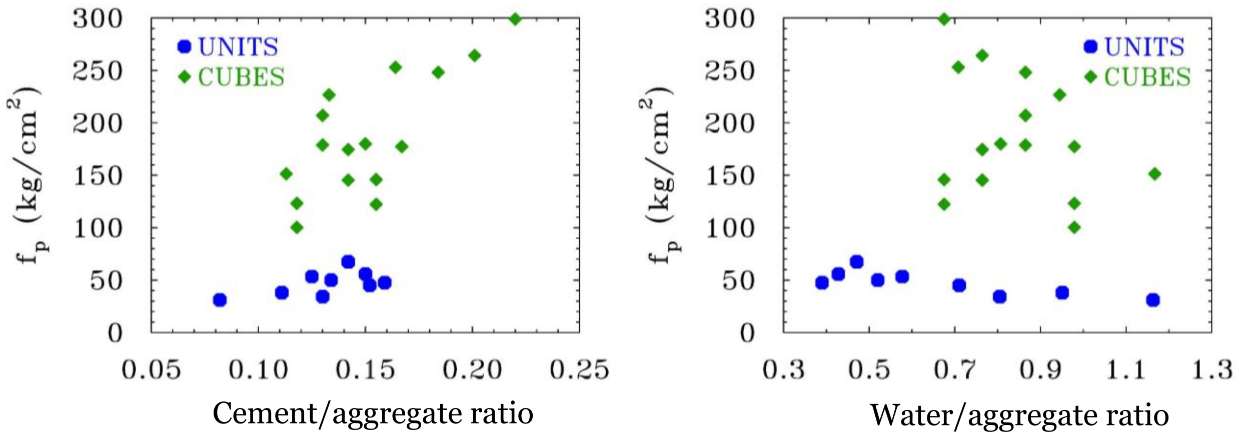
Results shown in Fig. 6 are for solid units and cubes manufactured with the same mix in the same date. The dependency of the compressive strength with respect to the cement/aggregate (Fig. 6a) and water/cement (Fig. 6b) ratios was evaluated. Fog curing was used for both cubes and concrete masonry units.



a) Cement/aggregate ratio b) Water/cement ratio

Figure 6. Dispersion of obtained compressive strength test results for cubes and solid masonry units made with tepojal as base material. Shown values correspond to the average of at least nine cubes and four solid units.

It can be observed from Figure 6 that the obtained compressive strengths for the solid units were low, independently of the cement/aggregate or water/cement ratios. Measured compressive strengths varied between 31.1 and 64.9 kg/cm^2 and their average values ranged between 47 to 54 kg/cm^2 (Fig. 6). No significant improvement on the measured compressive strength was observed as the cement/aggregate ratio increases (Fig. 6a) or the water/cement ratio decreases (Fig. 6a), particularly for the solid units, which were the objects of interest in this research. With respect to the water/cement ratio, perhaps the main reason of no observing an improvement as the water/cement ratio decreases is related with the great porosity and high water absorption characteristics of tepojal, and its inability to retain water in a controlled manner. Then, the resulting concrete mix may develop important shrinkage when loosing water at the time of drying, which does no favor an adequate chemical reaction with the cement and then, leading to a solid material that it is porous and weak. Therefore, unless the water/cement ratio is controlled to be 0.7 or smaller, when the higher compressive strength are obtained (Fig. 6b), the resulting mixes for higher water/cement ratios are not compact enough and the resulting solid units are not strong enough in the average. Therefore, it was concluded that tepojal most likely is not a suitable base material to produce quality concrete masonry units despite its popularity and availability, granted that the characteristics of the studied tepojal from the material bank of Toluca are representative of other tepojal banks in the valleys of Mexico and Toluca.



a) Cement/aggregate ratio   b) Water/cement ratio

Figure 7. Dispersion of obtained compressive strength test results for cubes and solid masonry units made with the tezontle-sand 30-70 mix as base material. Shown values correspond to the average of at least nine cubes and five solid units.

Given the discouraging results obtained with the tepojal mixes, then, it was decided to work with mixes using the tezontle-sand 30-70 mix in order to obtain a concrete with base aggregates of reasonable material properties. Average compressive strengths for cubes and masonry units made with these mixes and their relation with the cement/aggregate (Fig.7a) and the water/cement (Fig. 7b) ratios are shown in Fig. 7. Average values were obtained for 5 to 7 cubes or solid units. Different cement/aggregate ratios were initially studied (between 0.08 and 0.22, Fig. 7a) in order to try to obtain the objective (or target) average compressive strengths for the solid masonry units. As it is shown in Fig. 7a, no significant difference was observed in the average compressive strength obtained for the solid masonry units. However, an important difference was observed in the average compressive strength for the cubes. Strength increments in cubes for cement/aggregate ratios higher than 0.12 did not grow as rapidly as those obtained for smaller cement/aggregate ratios. Therefore, taking into account that solid masonry units rather than cubes are the main objects of interests of study, cement/aggregate ratios were finally adjusted to the following ratios: 0.053, 0.065, 0.084 and 0.120. This selection of mixes was done considering the manufacturing process and costs. It is not wise to augment the quantity of cement used in the mix if there are not evident increments in the compressive strength for the resulting concrete masonry units, because cement is the most expensive material used in the mix. A cheaper option would be always to improve the mix for base materials (aggregates). It is worth noting that the studied tezontle-sand 30-70 mix has good characteristics (although far from ideal ones), because the average compressive strength for the solid units increases (although slightly) as the water/cement ratio is reduced (Fig. 7b), as it should be expected when the water consumption is controlled in the manufacturing process. No workability problems were observed for mixes with water/cement ratios of 0.4 or smaller despite of not using any admixture. Obtained average compressive strengths for the solid masonry units varied from 47.5 kg/cm^2 for a water/cement ratio w/c=0.39 to 50 kg/cm^2 when w/c=0.52. The highest average compressive strength for the solid units was 67.43 kg/cm^2 when w/c=0.47. Obviously, a larger, more uniform and more controlled sampling of mixes with respect to the water/cement ratio is required to observe clear tendencies with respect to this variable; however, this was not the main objective of the conducted study. As mentioned earlier, the main purpose of this research was to define, in a practical way, four plausible concrete mixes to manufacture solid concrete masonry units with a fairly uniform distribution of their average compressive strength.

4. MANUFACTURING OF TEZONTLE-SAND 30-70 MASONRY UNITS

As described in the previous section, a massive manufacturing of solid concrete building bricks was done using a tezontle-sand 30-70 mix as base material, and cement/aggregate ratios of 0.053, 0.064, 0.084 and 0.120. This decision was done taking into account the experimental results reported in the previous section, as well as the opinion of the interested manufacturer that participated in this research, which informed this research team that production costs would considerably increase if the cement/aggregate ratio is higher than 0.12. The water/cement ratios used by the manufacturer varied from 0.43 to 0.50, taking into account the results of the reported tests (Fig. 7b).
The solid concrete building blocks were produced by the concrete block manufacturer (Fig. 8) using the mix proportioning provided by this research team, they were stowed with care and were cured under ambient conditions (Fig. 8b), adding water manually as required, which it is a typical manufacturing process of most small concrete block factories that operate in the Valley of Mexico. Of course, there are big industrialized concrete masonry units' manufacturers operating in the Valley of Mexico which do a careful selection of their material banks, a professional definition of their concrete mix proportioning and use highly industrialized manufacturing processes with low water consumption. Unfortunately, to the authors´ knowledge, there are only two industrialized manufacturers of concrete masonry units operating in the Metropolitan Area of Mexico City and its suburbs.

a) manufacturing of solid units b) stowage and curing of units

Figure 8. Manufacturing, stowage and curing of solid units made with tezontle-sand 30-70 mix

5. COMPRESSIVE STRENGTH OF TEZONTLE-SAND 30-70 BLOCKS

The compressive strength testing was done according to Mexican norm NMX-X-036 (2004), using the main testing machine of the Intermediate Structural Models Lab of UAM-A. The pitching of each solid concrete building brick was done using sulfur (Fig. 9a). Compressive testing was done at an established velocity of 1.3 mm/s up to the failure of the unit, which it normally was due to crushing and crumbling of the unit (Fig. 9c).

a) Pitching b) Testing c) Typical failure

Figure 9. Compressive test of solid concrete building bricks made with tezontle-sand 30-70 mix

Compressive strength tests were conducted for a set of each of the previously identified concrete mixes according to the guidelines established in NTCM-04 (2004). The compressive design strength for the units, f^*_p, is assessed as:

$$f_p^* = \frac{\bar{f}_p}{1+2.5c_p} \qquad (1)$$

where $\bar{f}_p$ is the mean compressive strength assessed over the gross area for the units and c_p is the coefficient of variation of the assessed compressive strength. Also, c_p shall not be less than 0.30 for mechanized production of units with no certified quality control, which is the description of the manufacturing process used in this study.

The results obtained for the testing of 10 units for each considered cement/aggregate ratio are reported in Table 1. In this table, c_p* is used to identified the minimum coefficient of variation established in NTCM-04 ($c_p^* \geq 0.3$) and f_{pp}* is the design compressive strength for the units if in NTCM-04 it would be allowed to use the coefficient of variation obtained from tests (c_p), even when it would be smaller than c_p*.

It is observed from Table 1 that as the cement/aggregate ratio increases, the average compressive strength $\bar{f}_p$ for the tezontle-sand 30-70 solid masonry building bricks increases and the coefficient of variation c_p notably decreases. The average compressive strength $\bar{f}_p$ were reasonably close to the target (objective) compressive strengths for cement/aggregate ratios equal or higher to 0.065, and they exceeded the expected target values for the 0.053 ratio. If c_p from tests could be directly used to assess the compressive design strength, plausible compressive design strengths could be obtained (f^*_{pp} values in Table 1). However, as per section 2.1.2 of NTCM-04 (2004), it is established that: “The value of c_p shall not be less than 0.20 for masonry units manufactured in factories with the quality control standard described in the norm NMX-C-404-ONNCCE, or 0.30 for masonry units manufactured in factories with no quality control, or 0.35 for hand-crafted manufactured masonry units”. Therefore, taking into account that the industrial production does not follow the quality control standard established in norm NMX-C-404-ONNCCE, in this case study c_p= c^*_p=0.30 should be taken and, therefore, the design compressive strength f^*_p is notably reduced for the manufactured concrete masonry bricks (a 19% to 46% reduction).

Table 1. Compressive design strength of solid masonry units made with a tezontle-sand 30-70 mix, according to NTCM-04 (2004)

Property	**Cement/aggregate ratio**			
	0.053	**0.065**	**0.084**	**0.012**
$\overline{f}_p$ (kg/cm$^{2)}$	92.63	108.86	133.26	186.48
c_p	0.18	0.19	0.08	0.08
f^*_{pp} (kg/cm^2)	63.88	73.80	111.05	155.40
c^*_p	0.30	0.30	0.30	0.30
f^*_p (kg/cm^2)	52.93	62.21	76.15	106.56

The spirit behind NTCM-04 is promoting the construction of higher quality masonry structures based upon experimental testing (at least masonry units and masonry prisms). From this viewpoint, in the humble opinion of the authors, the minimum values established for c_p should be carefully reviewed by NTCM code committee members using updated valued from recent tests for future versions of the guidelines, as those established in NTCM-04 are very conservative.

6. COMPRESSIVE STRENGTH OF MASONRY PRISMS

Once that it was possible to manufacture solid concrete building bricks with adequate compressive strengths using the tezontle-sand 30-70 mix, masonry prisms were built to assess the design compressive strength of masonry (f^*_m) according to NTCM-04.

6.1 Mortars

Mortars types I, II and III (Table 2) were proportioned in relative parts by volume as established in NTCM-04 (2004). A Portland cement classified as CPC 30R was used. Mortar mixing was done using a shovel under ambient conditions. The water added was the minimum required for workability, but taking into account also the initial rate of absorption of the manufactured concrete building bricks to favor chemical bonding. As expected, the lime used in mortars type I and II favored the chemical bonding with the masonry units.

Only for controlling information, but no to satisfy the strength requirements set in NTCM-04, up to six cubes of standard dimensions (Fig. 5b) were built to assess the compressive strength for each mortar. Mortar was poured in previously greased metallic molds to ease demoulding. The filling of molds was done as follows (Fig. 10a): a 1/3 of the mold was filled with mortar and then it was compacted with a ram tool using 25 strokes. This procedure was repeated for 2/3 of the mold and until the mould was completely filled. Once the mould was filled, it was leveled and the cube was placed in a humid room for a 24-hour fog curing. After this time frame, mortar cubes were demoulded and the curing was done under ambient conditions for the remaining of the 28 days. In fact, mortar cubes were placed together with the masonry prisms (Fig. 10b) which were simultaneously built, as described in the following section.

Compressive strength testing of mortar cubes was done after 28 days. For the testing, cubes were completely axially aligned to the plates of the testing machine but without using any pitching material (Fig. 10c). The velocity for the application of the axial load was 1.3 mm/s up to the failure of the cube. Given that the minimum of nine cubes established by NTCM-04 to assess their design compressive strength were not tested, only the average compressive strengths for six cubes are given for each mortar type for information purposes. Then, average compressive strengths for mortar were 115.2 kg/cm^2 for mortar type I, 89.3 kg/cm^2 for mortar type II and 41.7 kg/cm^2 for mortar type III.

Table 2. Relative parts by volume for the mortars used

Mortar type	Portland cement	Lime	Sand
I	1	-	3
II	1	1/2	3
III	1	1	3

a) fabrication b) curing and identification c) testing

Figure 10. Building, curing and testing of mortar cubes

The main reason of not being obsessed with the compressive strength for the mortar is that it is well-known that it is not its most important or main property. It is a common erroneous belief that it is better to have a strong mortar (high compressive strength) despite it may have a dry consistency because they are usually produced with higher cement consumption and no lime. Such dry mortars usually have chemical bonding problems with the masonry units, particularly for units with high initial rates of water absorption. Since the main property for a mortar in a masonry structures is to favor an adequate bonding with the units, it is much better to proportion mortar mixes that favor both a high water retentivity and the workability of the mortar. The basic “natural” ingredient to improve the water retentivity of the mortar is the lime, but since it reduces the compressive strength for the mortar, many people has the erroneous belief that is not good to add lime to a mortar. Lime is of paramount importance for a mortar mix with a high water retentivity, granted that one uses smaller amounts of lime than Portland cement to balance both water retentivity and an adequate compressive strength. This fact is well-known from a while ago, and that it is why in the masonry codes of the United States of America (for example, UBC-97, 1997; ACI-530, 2011), mortars for structural use are specified in relative parts by volume only and all mortars are required to have lime (Table 3). It is worth noting that in Table 3 no minimum compressive strength for the mortar is identified or even mentioned within the code, in contrast to what it is established in NTCM-04 (2004).

Table 3. Mortars for structural use specified in the masonry codes of the United States. Relative parts by volume

Mortar type	Portland cement	Lime	Sand
M	1	1/4	3 1/2
S	1	1/2	4 1/2
N	1	1	6
O	1	2	9

6.2 Masonry prisms

Masonry prisms were built according to what it is outlined in section 2.8 of NTCM-04 (2004). Each prism was made with five units joined with a mortar joint 1 cm thick; therefore, the height

of each prism was approximately 39 cm and its slenderness ratio is h/b=39/12.5=3.12, within the slenderness range established in section 2.8.1.1 of NTCM-04 ($2 \leq h/b \leq 5$).
Ten masonry prisms were made for each different concrete mix of the unit (4) and mortar type (3), resulting a total of 120 built prisms which were tested under axial compression. A working table was used for building the masonry prisms, using a thread line to help make sure the mortar joint had a thickness of 1cm ± 2mm, as established in Appendix A of norm NMX-C-404 (Figure 11). Masonry prisms were built by an experienced bricklayer who works in UAM-A.
The testing of masonry prisms was done after 28 days of being built, according to Mexican masonry regulations. Sulfur pitching was used for the masonry prisms (Fig. 12a). Compressive testing was also done as established in norm NMX-C-036. The observed failure for prisms started with the classical lateral tension splitting of the concrete bricks and the mortar joint (Fig. 12b), the desired failure mechanism in axial compression for a good masonry design: weak mortar – strong brick (for example, McNary and Abrams 1985).


a) Working table


b) thread line guides for mortar joints

Figure 11. Building of masonry prisms with solid concrete bricks made with the tezontle-sand mix

The compressive strength of each prism was determined according to NTCM-04 (2004), where the computed compressive strength has to be corrected using a prism slenderness correction factor (f_e) available in Table 2.5 of NTCM-04. Therefore, the individual compressive strength of each masonry prism (f_m) was assessed as:

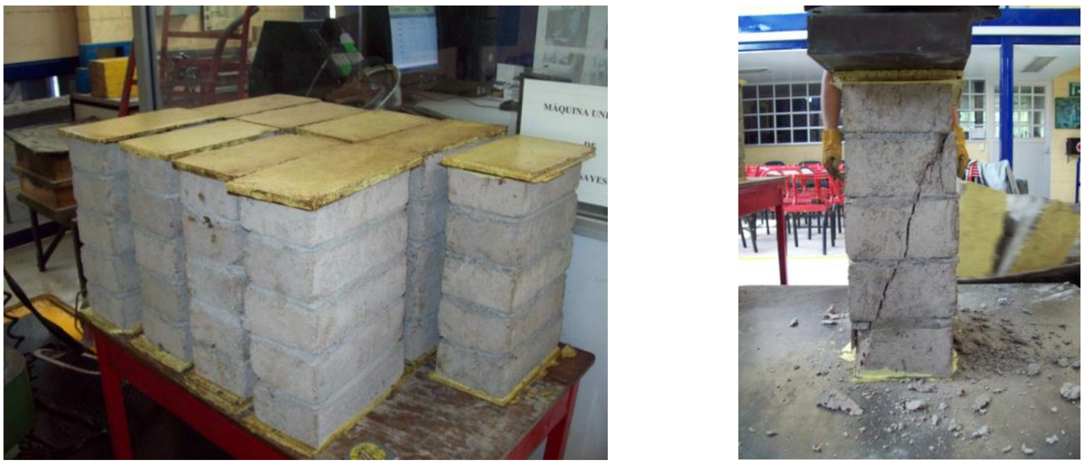
$$f_m = \frac{P}{A_n} f_e \tag{2}$$

where P is the applied axial load, A_n is the gross cross section area of the masonry prism and the slenderness correction factor for the prism was f_e=0.912 because the slenderness ratio is h/b=3.12. It is worth noting that before the pitching, four prisms of those joined with mortar type I (without lime, Table 2) had bonding problems in the last course, so for those prisms the top brick and mortar joint were removed before pitching, so four prisms with four units were tested and since the slenderness ratio was reduced to h/b=2.48, for those four prisms, f_e=0.807. Given that only 4 out of 40 prisms were affected, and it was practically only one for each type of solid concrete brick (in terms of the cement/aggregate ratio), it was considered that including the results obtained for these 4 prisms did not significantly affect the statistics of the prism testing discussed herein. Nevertheless, this incident allow one to help illustrate again that bonding problems tend to occur more frequently in mortars without lime. For other mortar types that contain lime (Table 2) there was only one prism where a bonding problem occurred at the last brick, and it was using mortar type III (Liga and Pérez, 2013).

The compressive design strength for the masonry from prism tests, f_m*, was assessed according to NTCM-04 (2004) as:

$$f_m^* = \frac{\overline{f}_m}{1+2.5c_m} \tag{3}$$

where $\overline{f}_m$ is the average compressive strength for the prisms (at least nine are required, ten were used in this study) and c_m is the coefficient of variation of the tested prisms, which shall not be smaller than 0.15 (c_m*=0.15).



a) Preparation of prisms for testing b) Typical failure

Figure 12. Compressive strength testing for the masonry prism under study

The obtained testing results for each mortar type and cement/aggregate mix used for the concrete units are reported in Tables 4 to 6. In those tables, c_m* is used to identify the minimum coefficient of variation that must be used for design purposes according to NTCM-04 ($c_m^* \geq 0.15$), and f^*_{mp} is used to identify the design compressive strength for the masonry if in NTCM-04 it would be allowed to use the coefficient of variation obtained from prism tests (c_m) when is smaller than c_m*.

It is observed from the results shown in Tables 4 to 6 that the coefficient of variation obtained from prism tests, c_m, was always smaller than the minimum value c_m*=0.15 established in NTCM-04. In general, the highest experimental c_m was obtained for the building bricks produced with the smallest cement/aggregate ratio of 0.053, except for mortar type II (Table 5). In general, the coefficients of variation obtained experimentally for the remaining cement/aggregate ratios for the concrete bricks varied from 42% to 72% the minimum value c_m*=0.15 established in NTCM-04. Therefore, as a consequence, the obtained compressive strengths f^*_{mp} are between 8% to 20% higher than NTCM-04 design compressive strength f^*_m for the mortars under study for concrete bricks manufactured with cement/aggregate ratios of 0.065 or higher. Perhaps in the case of prisms the fact that in NTCM-04 a minimum coefficient of variation of 0.15 is set does not lead to spectacular differences in the design compressive strength f^*_m when compared to those obtained using the coefficient of variation obtained from testing (f^*_{mp}). Nevertheless, for consistency, this minimum value c_m*=0.15 should also be revised to help promoting the design of masonry structures based upon experimental data.

Table 4. Design compressive strength of masonry prisms joined with mortar type I

Property	**Cement/aggregate ratio**			
	0.053	**0.065**	**0.084**	**0.012**
$\bar{f}_m$ (kg/cm^2)	29.42	40.54	53.02	97.89
c_m	0.145	0.10	0.08	0.07
f^*_{mp} (kg/cm^2)	21.58	32.36	44.56	83.81
c^*_m	0.15	0.15	0.15	0.15
f^*_m (kg/cm^2)	21.40	29.49	38.56	71.19

Table 5. Design compressive strength of masonry prisms joined with mortar type II

Property	**Cement/aggregate ratio**			
	0.053	**0.065**	**0.084**	**0.012**
$\bar{f}_m$ (kg/cm^2)	30.72	38.33	51.64	96.20
c_m	0.08	0.06	0.08	0.11
f^*_{mp} (kg/cm^2)	25.70	33.32	43.36	75.83
c^*_m	0.15	0.15	0.15	0.15
f^*_m (kg/cm^2)	22.34	27.88	37.55	69.97

Table 6. Design compressive strength of masonry prisms joined with mortar type III

Property	**Cement/aggregate ratio**			
	0.053	**0.065**	**0.084**	**0.012**
$\bar{f}_m$ (kg/cm^2)	31.50	37.49	50.82	89.22
c_m	0.12	0.09	0.09	0.07
f^*_{mp} (kg/cm^2)	24.43	30.33	41.38	76.46
c^*_m	0.15	0.15	0.15	0.15
f^*_m (kg/cm^2)	22.91	27.76	36.96	64.89

7. COMPARISON WITH NTCM HISTORIC DESIGN TABLE

It was established since the first version of the Masonry Design Guidelines of Mexico´s Federal District Code published in 1977 (NTCM-77, 1977) to the 1995 version (NTCM-95, 1995), that the compressive strength for design of masonry, f^*_m, shall be taken from Table 7 if concrete masonry units satisfied that their slenderness aspect ratio is $\frac{h}{t} \geq 0.5$, their design compressive strength is $f^*_p \leq 200$ kg/cm^2. Besides, the concrete masonry units and the mortar should comply respectively with the quality control requirements established in sections 2.1 and 2.2 of those documents.

Table 7. Design compressive strength of masonry made with concrete units (NTCM-77 to 95)

f^*_p (kg/cm^2)	f^*_m (kg/cm^2)		
	Mortar type I	**Mortar type II**	**Mortar type III**
25	15	10	10
50	25	20	20
75	40	35	30
100	50	45	40
150	75	60	60
200	100	90	80

For the 2004 version of the guidelines (NTCM-04, 2004), the rows related to design compressive strengths for the concrete units below 100 kg/cm^2 were excluded (f^*_p<100 kg/cm^2). This was done because, at the times, the masonry code committee had information that most of the produced and marketed concrete masonry units were of poor quality (reduced design compressive strengths). The design shear strength v^*_m obtained from diagonal compression wallet tests established in Table 2.8 of NTCM-04 (2004) was obtained in the experimental program of the 1960s-70s for concrete units where $f^*_m \geq 85$ kg/cm^2 (Hernández, 1999), as it can be deducted from the observation of the histogram data depicted in Fig. 1. Therefore, since the seismic risk and hazard of the Metropolitan Area of Mexico City is high and then, the seismic design of masonry walls is ruled by shear, at the times, the masonry code committee decided to established a minimum quality for the concrete units ($f^*_p \geq 100$ kg/cm^2) to warrant the proposed shear strength v^*_m.

It is worth noting that the historic design table of NTCM (Table 7) is exclusively retaken with the only purpose of comparing in a more ample interval the obtained results in this experimental study, as all the test results obtained during the 1960s-70s are synthesized in this table. The authors do not want to promote that people would obtain “design strengths” by employing low quality concrete masonry units (f^*_p<50 kg/cm^2), which would not allow engineers to warrant the integrity of the structure and more importantly, the safety of people that uses such structures. Engineers who built masonry structures with such masonry units with an absolute knowledge of their poor quality lack any social commitment and ethical conduct.

Design curves f^*_p vs f^*_m according to NTCM (Table 7) are identified with full circles in Fig. 13 and compared to those obtained in this study, considering both the minimum coefficients of variation c^*_p=0.30 and c^*_m=0.15 established in NTCM-04 (2004) and the coefficients of variations c_p and c_m assessed experimentally in this study for the concrete masonry units (Table 1) and the masonry prisms (Tables 4 to 6).

It is observed in Fig. 13 that the curves for the experimental coefficients of variation c_p and c_m (full triangles) are similar to the curves related to the historic design tables of NTCM (full circles, Table 7) for the strength range of f^*_p where they coincide. In contrast, if one uses the minimum coefficients of variation c^*_p=0.30 and c^*_m=0.15 established in NTCM-04 (full inverted triangles), the obtained curves are not similar to those of the design tables of NTCM (full circles), as they exhibit a much higher slope. The reason behind it is that c^*_p=0.30 is much higher than c^*_m=0.15 and, therefore, the compressive design strength for the concrete masonry units (f^*_p) is reduced more than the compressive design strength for the masonry (f^*_m), as previously discussed. Then, as currently established in NTCM-04 for the design based upon experimental data (using c^*_p=0.30 and c^*_m=0.15), one would think that higher compressive design strengths for the masonry f^*_m are developed for concrete masonry units with $f^*_p \geq 75$ kg/cm^2 (full inverted triangle symbols). However, if the coefficients of variation obtained from testing c_p and c_m are used (full triangle symbols), it seems that the tendency is similar to the one obtained during the 1970s, which it seems more congruent.

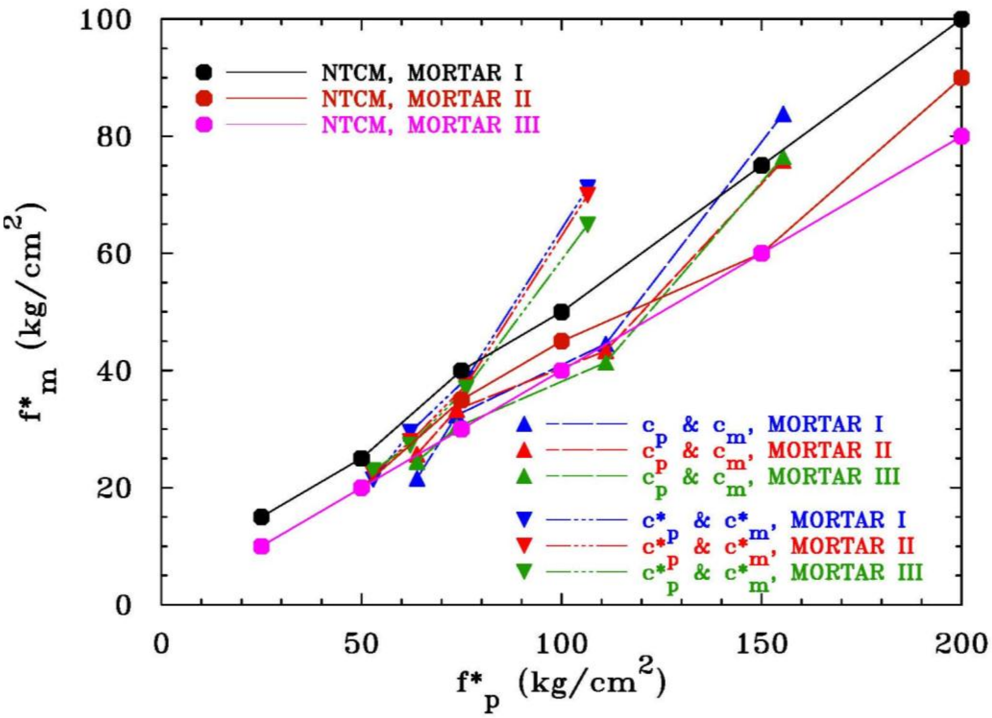


Figure 13. Compressive design strength for the masonry (f^*_m) vs compressive design strength for the concrete masonry units (f^*_p). Comparison of the obtained values in this experimental study with those established in NTCM historic table

In light of the obtained and discussed results, it seems very conservative to use a minimum coefficient of variation c^*_p=0.30 to assess the compressive design strength f^*_p for the masonry units. It is observed in Table 1 that c_p values obtained from tests varied from 0.08 for the stronger concrete bricks to 0.19 for the weaker concrete bricks. Then, and still taken a conservative criterion, it was decided to evaluate how the curves would look like if a fixed value c_p=0.20 was used. In fact, as stated earlier, this value corresponds to masonry units manufactured in factories with the quality control standard described in the norm NMX-C-404-ONNCCE, that is, with the highest quality control, which was not the procedure used to manufacture the studied concrete bricks. Likewise, and taking into account that the maximum coefficient of variation c_m obtained from testing was 0.145 (Table 4), a fixed value c_m=0.15 was used, which coincides with the one currently proposed in NTCM-04. The resulting f^*_p vs f^*_m design curves are compared to those corresponding to NTCM historic table (Table 7) in Fig. 14. A reasonable correlation is observed among the plotted NTCM (full circles) and experimental (full squares) curves for the compressive strength range of f^*_p where they coincide. However, smaller differences are observed for the obtained curves using the proposed c_p and c_m values (full squares) among mortars type I to III (particularly between mortars type II and III) than those observed in NTCM curves (full circles). It is also observed that as the compressive strength of the concrete masonry units (f^*_p) increases, a wider difference in the associated f^*_m values are observed for each mortar (curves start to separate more, particularly with respect to the weakest mortar).

Then, taking the results obtained with c_p=0.20 and c_m= 0.15 to make a closer proposal to the current philosophy established in NTCM-04, the values proposed in Table 8 could be used for design purposes for the studied concrete building bricks, being already rounded to practical and conservative values. In fact, for the design compressive strength (f^*_p) range studied for the concrete building bricks, it is not worth distinguishing the design compressive strength for the masonry (f^*_m) for mortar types I and II, but it does for mortar type III (Fig. 14). The minimum design compressive strength for the concrete masonry units $f^*_p \geq 60$ kg/cm^2 for structural use established in the norm NMX-C-404-ONNCCE is already considered in Table 8.

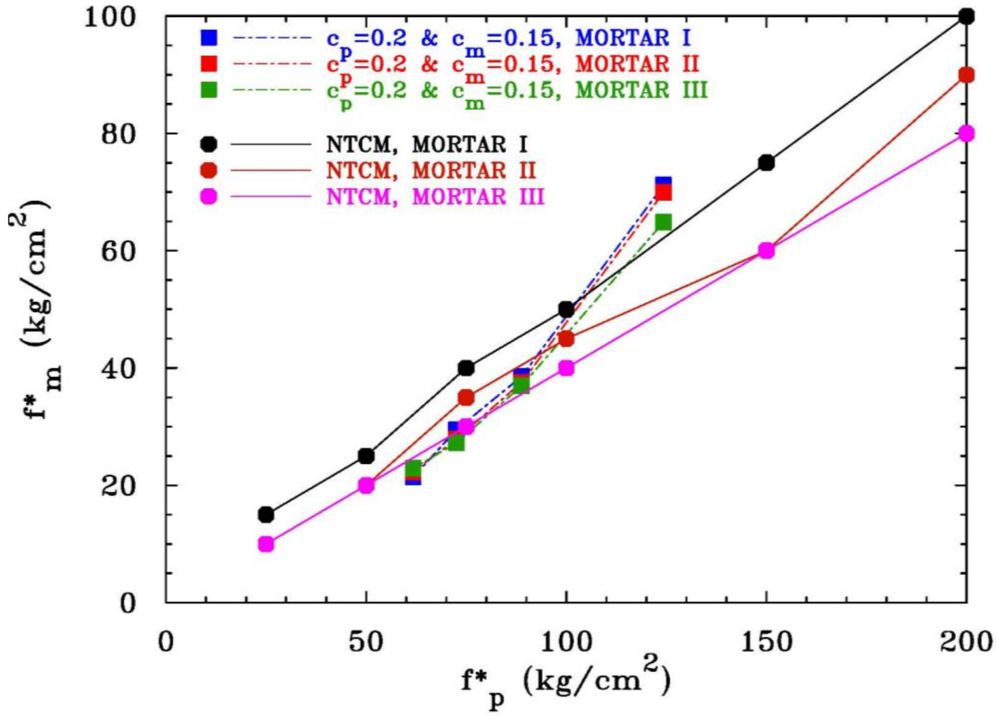


Figure 14. Compressive design strength for the masonry (f^*_m) vs compressive design strength for the concrete masonry units (f^*_p). Comparison of the proposed design values for the concrete bricks under study with those established in NTCM historic table

Table 8. Design compressive strength of masonry for solid concrete building bricks manufactured with a tezontle-sand 30-70 mix

f^*_p (kg/cm^2)	f^*_m (kg/cm^2)		
	Mortar type I	**Mortar type II**	**Mortar type III**
60	20	20	20
75	30	30	28
100	48	48	45
125	70	70	65

8. CONCLUDING REMARKS

An experimental study was conducted where solid concrete building bricks were manufactured using the base material (aggregates) commonly used in the Valley of Mexico by different small and medium size manufacturers. Different lab tests were done to the most commonly used aggregates: tepojal, tezontle and common sand. It was conclude from lab tests that tepojal (at least the one studied from the material bank in Toluca) is not a suitable base material to produce quality concrete masonry units despite the amount of cement used in the concrete mix.
For this reason, a tezontle-sand 30-70 mix (in volume) was designed and used, as it allowed to develop concrete mixes with much better compressive strength and water absorption properties.

Then, four different cement/aggregate ratios were defined to elaborate concretes with the tezontle-sand 30-70 aggregate mix. The obtained average compressive strength for the concrete obtained with those mixes varied from 90 to 190 kg/cm^2 with coefficients of variation that ranged between 0.08 and 0.19, much smaller values than the minimum coefficient of variation c_p=0.30 established in NTCM-04 for masonry units manufactured in factories with no quality control.
Masonry prisms were built for each mortar type specified in NTCM-04 (mortar types I, II and III) and later on tested under axial compression. Design compressive strengths for the masonry f^*_m as established by NTCM-04 were assessed from the experimental results, using both the coefficients of variation obtained from the testing, as well as the minimum coefficient of variation established in NTCM-04. It is worth noting that the coefficients of variation obtained experimentally were within the range $0.06 \leq c_m \leq 0.145$ and therefore, they were always smaller than the minimum value established in NTCM-04 (c_m=0.15).
If one rigorously applies what it is established in NTCM-04 to assess the design compressive strengths f^*_p for the masonry units and f^*_m for the masonry, the resulting design curves have steep slopes that do not compare well with the design curves related to the design tables proposed in NTCM-04. Therefore, it is important to revise in NTCM the minimum values proposed for the coefficient of variation for the masonry units, c_p, because they seem to be excessively conservative according to the experimental values obtained in this study. It would be also desirable to review the minimum value proposed for the coefficient of variation for the masonry prisms, c_m, although this latest minimum value seem to be much more reasonable when compared to the obtained experimental data.

9. ACKNOWLEDGMENTS

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Ageing of old and modern concrete structures – Observations and research

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ABSTRACT

Ageing is an inherent feature of nature. Yet it seems to be a rather new topic in both science and engineering. The main reason for increasing attention for ageing as a topic is the growing awareness that, particularly in industrialized countries, ageing of our assets is a financial burden for the society and affects the overall sustainability of our planet. In this contribution, the urgency and challenges of ageing of concrete structures are addressed. The complexity of ageing problems will be illustrated by looking in more detail to the evolution in concrete mix design and the consequences thereof for the long-term performance of concrete structures. Emphasis will be on ageing of concrete infrastructure and justification of research on ageing phenomena.
Keywords: infrastructure; sustainability; ageing; mix design; autogenous shrinkage; codes.

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Envejecimiento de antiguas y modernas estructuras de concreto - Observaciones e investigaciones

RESUMEN

El envejecimiento es una característica inherente de la naturaleza. Sin embargo, parece ser un tema bastante nuevo en la ciencia y la ingeniería. La principal razón para aumentar la atención por el envejecimiento como tema es la creciente conciencia de que, en particular en los países industrializados, el envejecimiento de nuestros activos es una carga financiera para la sociedad y afecta la sostenibilidad global de nuestro planeta. En esta contribución se abordan la urgencia y los desafíos del envejecimiento de las estructuras de concreto. La complejidad de los problemas de envejecimiento se ilustra examinando con más detalle la evolución del diseño de la mezcla de concreto y sus consecuencias para el rendimiento a largo plazo de las estructuras de concreto. Se hace hincapié en el envejecimiento de las infraestructuras de concreto y en la justificación de la investigación sobre fenómenos de envejecimiento.

Palabras clave: infraestructura; sostenibilidad; envejecimiento; mezcla de diseño; contracción autógena; códigos.

Envelhecimento de estruturas de concreto antigas e modernas - Observações e pesquisas

RESUMO

Envelhecimento é uma característica inerente da natureza. No entanto, parece ser um tópico bastante novo tanto na ciência quanto na engenharia. A principal razão para aumentar a atenção para o envelhecimento como tema é a consciência crescente de que, particularmente nos países industrializados, o envelhecimento de nossos bens é um fardo financeiro para a sociedade e afeta a sustentabilidade global do nosso planeta. Nesta contribuição, são abordados a urgência e os desafios do envelhecimento das estruturas de concreto. A complexidade dos problemas de envelhecimento será ilustrada por uma análise mais detalhada da evolução da concepção da mistura de concreto e suas consequências para o desempenho em longo prazo das estruturas de concreto. A ênfase será no envelhecimento da infraestrutura de concreto e na justificativa da investigação sobre fenômenos de envelhecimento.

Palavras-chave: infraestrutura; sustentabilidade; envelhecimento; estudo de dosagem; retração autógena; norms.

1. INTRODUCTION

Ageing is everywhere around us. Huge mountains seem to keep their shape for ever. But at a closer look we see that the surface of rocks gradually changes. Changes in temperature and moisture conditions, wear, wind and light are sufficiently powerful to crumble even the strongest rock. Mountains age! Earthquakes may split mountains, causing changes in the state of stress in the newly formed parts of the mountain. Fresh fracture surfaces become exposed to climatic conditions and another cycle of ageing starts.
Like rocks, also man-made structures are exposed to ambient climate conditions. While exposed to environmental loads, structures must carry life loads and deadweight in a safe way during their entire service life. Roads and railways need continuous maintenance. If planned correctly, the trouble maintenance works often cause can be kept to a minimum. If maintenance comes too late, expensive repair is needed and may cause time and money consuming traffic jams, delays or even

accidents. The direct costs of failing infrastructure can be huge, but the indirect costs are generally many times higher.
Proper functioning of our infrastructure is vital for the country's mobility and economy. The same holds for our energy infrastructure. Power plants for generating electricity and energy distribution grids must function reliably for 24 hours per day, the whole year round. Failing components may cause expensive process interruptions and may even constitute a risk for life and limb. Pro-active replacement of vital components of systems and structures is considered a safe strategy to prevent catastrophic failures. But do we really know how close we were to a catastrophic failure at the moment these components were replaced? Was the society really at risk or did we spoil a lot of still perfectly operating components without improving safety substantially? In other words: how accurately can we predict the progress of ageing processes from which our assets are suffering?
Ageing is everywhere and unavoidable. Yet it is not easy to find a clear an unambiguous definition of ageing. The term ageing is used for changes in performance with time of materials, structures, systems, organisations, societies, governments, software, economic systems, living organisms, etc. These changes in performance can be observed at different scales. But what are the real driving forces behind these changes? Before starting an attempt to explain what we mean by ageing, we first give an impression of the societal relevance of ageing of our fixed assets, with a focus on ageing of our infrastructure.

2. AGEING INFRASTRUCTURE AND SOCIETY

In modern industrial countries, the infrastructure makes out over 50% of the nation's national wealth (Long, 2007). This infrastructure consists of roads and railway systems, water works, airports, power stations and electricity grids. Based on an inventory in twelve countries, the value of infrastructure stock averages around 70% of the global gross domestic product (GDP). For a global GDP of € 53 trillion in 2012, this makes € 37 trillion.
Economic growth is inconceivable without growth of a country's infrastructure. To catch up with global economic growth the McKinsey report (Dobbs et.al., 2013) estimates a required investment in the infrastructure of € 42 trillion between 2013 and 2030. This means an annual investment of € 2.3 trillion, which is about 4.5% of the global GDP. The investment of € 42 trillion is needed for roads and railways, ports, airports, power stations, water works and telecommunication. Table 1 gives the breakdown of investments over these categories. These figures are (in part) based on an extrapolation from data provided by 84 countries.

Table 1. Estimated needs for global infrastructure in different categories. Period 2013-2030 (Dobbs et.al., 2013)

Category	Source	Required investment [× € 1,000,000,000,000]
Roads	OECD[1])	12.2
Rail	OECD	3.3
Ports	OECD	0.5
Airports	OECD	1.4
Power	IEA[2])	8.8
Water	GWI[3])	8.4
Telecommunications	OECD	6.8
Total		41.4

1) Organisation for Economic Co-operation and Development
2) International Energy Agency
3) Global Water Intelligence

These countries are responsible for 90% of the global GDP and are considered today's best possible basis for estimating the extra investments needed for our infrastructure in the period from 2013 to 2030.

3. AGEING AND SCIENCE

3.1 Change of performance with time.

The early lifetime of made-made materials, structures and systems is often characterized by a high probability of failure. It takes some time to overcome inevitable teething problems and to reach the required level of maturity and stability. Once that point is reached a 'quiet' period follows until we arrive again at a period of increasing probability of failure. Exceeding a certain predefined probability of failure then marks the end of the service life of a structure or system. The high probability of failure in the beginning, the subsequent period of 'rest' and the subsequent period of increasing probability of failure can be presented with the bathtub curve (Figure 1a).

In essence the bathtub curve also applies to our fixed assets, even though it has not very often been used for infrastructure. The length of the period in which the probability of failure is low is of crucial importance for the economic performance of these assets. The bathtub curve suggests that this period is a period during which 'nothing happens'. It is a period of 'rest', or 'dormant' period. Assuming that in the period of low probability of failure nothing happens, however, is misleading. If there would really be 'rest', what could then be the driving force behind the increase of probability of failure with elapse of time? To illustrate the foregoing reasoning, it may help to put the bathtub curve of Figure 1a upside down, as shown in Figure 1b. On the vertical axis we now put 'Performance' instead of the 'Probability of failure'. After a short period of teething problems the material, structure or system has reached the required (high) level of performance. That is the level at which the material should demonstrate its capacity to meet safety and functional criteria, if possible without intervention for maintenance or repair. It is the period of 'top-level sport' for all the basic building blocks, i.e. atoms, molecules and interfaces,

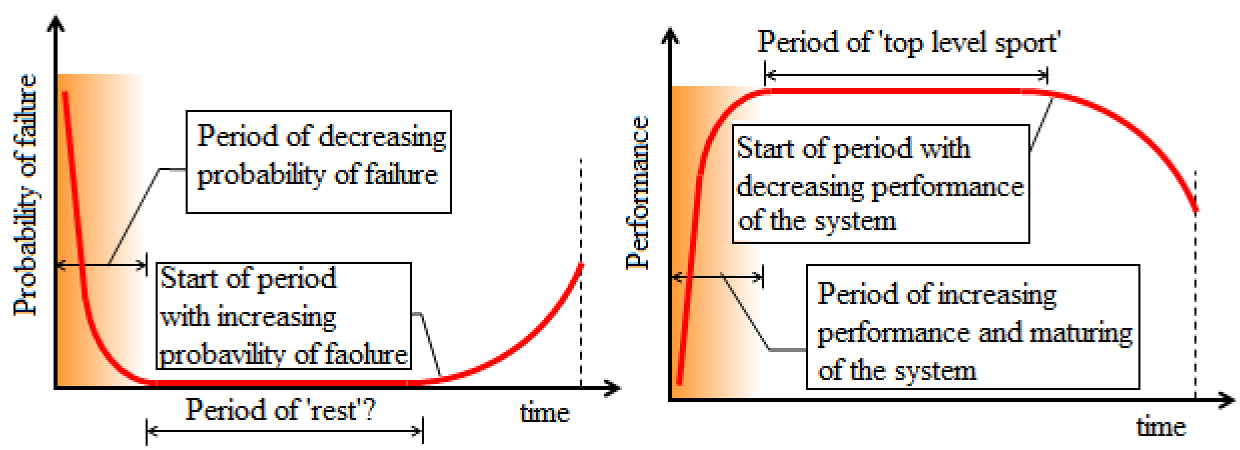

Figure 1. Evolution of the probability of failure (bath tube curve; left) and of performance (right) of complex systems (after Van Breugel, 2014)

from which a material or structure is made. When these basic building blocks give up and leave their position, the period of decay begins. Then ageing has started! These first tiny steps of decay will most probably not be observed at the macroscale immediately.

The moment that the first basic building blocks give up can only be captured with comprehensive and appropriate material models at subsequent levels of observation. Here chemistry, physics, electrochemistry, mechanics and mathematics *meet* each other and *need* each other for developing tools for describing and predicting ageing processes at a fundamental level.

3.2 Driving forces behind ageing – A closer look.

Ageing has been defined is a change of performance of a material, structure or system with time. How time *per se* can result in a change of performance is not easy to understand at first sight. How can a material 'at rest' change its performance with time? A closer look at any piece of matter 'at rest' tells us that the status of rest only applies to a certain length scale. Going down to the atomic scale the world is in motion all the time! Fundamental entities, i.e. basic building blocks, are continuously moving with a certain probability to leave their position for one that fits them better. This phenomenon takes place in the time domain. It is an inherent feature of matter and lies at the basis of ageing of materials. On top of this inherent feature we see, at different scales, a number of *gradients*, which may cause the basic building blocks of matter to start moving. Gradients are the driving forces causing changes in the material with elapse of time. Note that at the boundary of any piece of material with its environment gradients do exist. These gradients concern, for example, temperature, humidity and radiation and they may cause changes at the surface of the material.
The foregoing illustrates that a material 'at rest' is hardly conceivable. Al lower scales there is motion all the time and a variety of gradients cause the basic building blocks of matter to change their position. In essence, this holds for all materials and systems. Basic building blocks strive for a position (energy level), where they feel more comfortable. By designing materials in a smart way, i.e. by minimizing internal gradients and concentrations of stresses and strains, there will be less reason for basic building blocks to leave their position. Hence, the ageing process will slow down and the service life of materials, structures and systems will increase.

4. PERFORMANCE OF CONCRETE STRUCTURES

4.1 Performance of bridge decks – An inventory.

In March 2001, the results of a most interesting study were published by Mehta (Mehta and Burrows, 2001). He analysed the performance of bridge decks of bridges built in four subsequent periods in the twentieth century. The first period was the period before 1930, the second between 1930 and 1950, the third from 1950 to 1980 and the fourth from 1980 to present. The concrete mixtures used for the bridge decks were characterized by the chemical composition and the fineness of the cement. The cements used in the first period, before 1930, had a C_3S content less the 30% and a Blaine surface of 180 m^2/kg. Consequently, the rate of hydration was low. The performance of many of the bridge decks made with these cements was quite good.
The cements used in the second period were ground to a Blaine fineness between 180 and 300 m^2/kg. The construction and building technology used for the bridge decks was similar to those used in the first period. The authors report that the bridge decks built in the second period were less durable than those built before 1930.
The structures that were built between 1950 and 1980 appeared to have more durability problems than those built before 1950. The cements used in this period had a fineness up to 400 m^2/kg and a C_3S content beyond 60%. With the aim to get a denser and more durable concrete the w/c ratio was lower than in the first two periods. The higher C_3S content and the higher fineness of the cement had increased the early strength of these mixtures. This made it possible to build faster. This, however, had resulted in a higher probability of early-age thermal cracking and, on top of that, higher autogenous shrinkage of the low water-cement ratio mixtures. The higher proneness to early-age (micro)cracking was the most plausible reason for durability problems at later ages.

In the fourth period the tendency to go for higher strengths continued. Generally, this was realized by using mixtures with a low w/b ratio. The use of low w/b mixtures further increased the risk of cracking. For bridge decks, moderate strengths between 30 and 45 MPa were found. Among 29 bridge decks the cracking in 44 MPa bridge decks was twice that in 31 MPa bridge decks.

4.2 Mix design and proneness to ageing.

Mehta's study of the performance of bridge decks illustrates how the pressure from the market to build faster has created a demand for mixtures with a high early strength. This was possible by using finer cements with a higher C_3S content. The price of this, however, was a higher probability of early-age cracking of the bridge decks.

For realizing slenderer and more elegant structures a higher final strength is required. High strength is attainable by reducing the w/b ratio. The use of (super) plasticizers has made it possible to reduce the w/b ratio of concrete mixtures to values even below 0.2. With these low w/b mixtures dense concretes are obtained with low permeability. This is considered good for the concrete's durability. At the same time, however, we see an increase in the concrete's proneness to (micro)cracking, mainly because of increased autogenous shrinkage.

Another reason for a higher cracking risk of high strength and ultra-high strength concretes are the high temperatures that occur because of high cement contents. By optimizing the particle packing of the aggregate fractions the amount of cement, and hence the peak temperatures, can be reduced. A low cement content is also considered positive from the sustainability point of view (lower carbon footprint of the fresh concrete mixture). A low cement content, however, also has a drawback. A low cement content reduces the inherent self-healing capacity of the concrete. From the self-healing point of view a not too low cement content and the use of 'old', coarsely ground cement is favorable. This partly explains the outcome of Mehta's study that old bridge decks performed better than newer ones. In the terminology of this paper we would say that the old concrete mixtures with coarse cements with a low C_3S content were less prone to ageing than modern mixtures with finely ground cements with a high C_3S content.

5. AUTOGENOUS SHRINKAGE – A CLOSER LOOK

For understanding ageing of traditional and modern concrete mixtures we need a clear picture of the processes that cause internal stresses in the material. As discussed in section 3 these internal stresses are among the driving forces of ageing. One of the causes of internal stresses is autogenous shrinkage of hardening concrete. In this section, experimental results of autogenous shrinkage of traditional and high strength concrete mixtures are presented, as well as technological measures to mitigate autogenous shrinkage. The measured autogenous shrinkage will be compared with values given by currently used design codes.

5.1 Shrinkage – Influencing factors.

Several mechanisms have been proposed as possible causes of autogenous shrinkage and/or contributing factors. The most commonly reported mechanisms are capillary tension (in the range of high internal relative humidity), changes in the disjoining presses (in medium range relative humidity) and changes of surface tension of the solid gel particles. A common parameter in all these mechanisms is the internal relative humidity. With progress of the hydration process the water in the mixture is gradually consumed and the relative humidity drops. This so-called 'internal drying' goes along with an increase in the capillary pressure in the pore water and, when the relative humidity decreases further, changes in the disjoining pressure. Consequently, the volume of the cement paste decreases, which is known as autogenous shrinkage.

Shrinkage of the drying paste is restrained by the aggregate particles in the mixture. Whether the restraint of autogenous shrinkage strains will cause (micro)cracking depends on the size and stiffness of the aggregate particles and on the time dependent properties (creep, relaxation) of the hardening paste. How *internal curing* of concrete mixtures can be used to prevent a drop of the relative humidity, and hence of autogenous shrinkage, will be discussed in the next section.
The fact that the evolution of autogenous shrinkage appears to be strongly correlated with a drop in the internal relative humidity does not mean that the magnitude of autogenous shrinkage can be related directly to the relative humidity. The type of cement has turned out to be an important parameter as well (Tazawa and Miyazawa, 1997). In the very early stage of hydration some types of cement exhibit swelling. This observation is of utmost importance if it comes to the interpretation of shrinkage measurements. Researchers should be aware of the fact that, particularly in the early stage of the hydration process, measured shrinkage strains are the net result of simultaneous expansion and shrinkage mechanisms. In those cases, attributing measured shrinkage strains to one single mechanism will lead to completely wrong conclusions about the underlying mechanisms and, consequently, to wrong measures to prevent or mitigate autogenous shrinkage.
From this brief overview, we learn that a number of parameters affect the magnitude of autogenous shrinkage. By manipulating these parameters, the consequences of autogenous shrinkage can be mitigated and hence the susceptibility of concrete mixtures to ageing. In the next sections emphasis will be on autogenous shrinkage and internal curing of HPC and how internal curing reduces autogenous shrinkage.

5.2 Autogenous shrinkage in C55/65 mixtures and internal curing.

As indicated in the previous section, autogenous shrinkage of low w/c mixtures can be reduced by internal curing. Internal curing can be accomplished by adding water-saturated lightweight aggregate (LWA) particles to the concrete (Zhutovsky et.al., 2001). When the internal RH drops, the water stored in the LWA particles is released to the drying matrix, thus maintaining the RH at a relatively high level. A similar effect can be achieved with mixed-in super absorbing polymers (SAP), a technology promoted by Jensen (2013) and subject of the RILEM committee 225-SAP (Mechtcherine and Reinhardt, 2012).
In the following results of studies on autogenous shrinkage of concrete and the effect of internal curing will be presented. First test results on autogenous shrinkage of concrete mixtures C55/65 are discussed, followed by results obtained with mixtures C28/35 and C35/45.
5.2.1 Background of the study. In the eighties and nineties of the past century the use of High Performance Concrete (HPC) with target strength C55/65 was considered for several concrete bridges in The Netherlands. The prevailing Dutch design code did not require designers to consider autogenous shrinkage of those mixtures. However, the owner of the bridges, the Dutch Ministry of Transportation, required a check of the overall performance of the mixtures, including a check of the autogenous shrinkage and the effectiveness of internal curing for mitigating the risk of early-age cracking.
5.2.2 Mixture design and test specimen. Four mixtures were tested with w/b ranging from 0.34 to 0.39. The mix compositions are given in Table 2. In mixture I, 60 kg limestone powder was used, while the amount of cement was reduced by the same amount. In mixture IV, 25% of the coarse aggregate was replaced by water-saturated lightweight aggregate, Liapor F10. The autogenous shrinkage was measured on sealed specimens, 100×100×400 mm^3.

Table 2 Mixture compositions of HPC (C55/65) (Van Breugel et.al., 2000)

Component	Mixture				
	Unit	I	II	III	IV
Water	kg/m^3	133	153	156	156
CEM III/B 42.5 LH HS	kg/m^3	248	340	300	300
CEM I 52.5 R	kg/m^3	112	110	100	100
Limestone powder[1])	kg/m^3	60	--	--	--
Water/cement ratio		0.37	0.34	0.39	0.39
Sand 0 – 4 mm	kg/m^3	942	860	830	830
Crushed aggregate 4–16 mm	kg/m^3	997	980	975	730
Liapor F10, 4-8 mm	kg/m^3	--	--	--	156
HR Superplast. CON 35	kg/m^3	5.0	--	--	--
Cretoplast CON 35	kg/m^3	--	1.8	--	--
Cretoplast SL01 CON 35	kg/m^3	--	7.2	--	--
Addiment BV1	kg/m^3	--	--	1.6	1.6
Addiment FM 951	kg/m^3	--	--	4.8	4.8

1) Fineness 530 m^2/kg

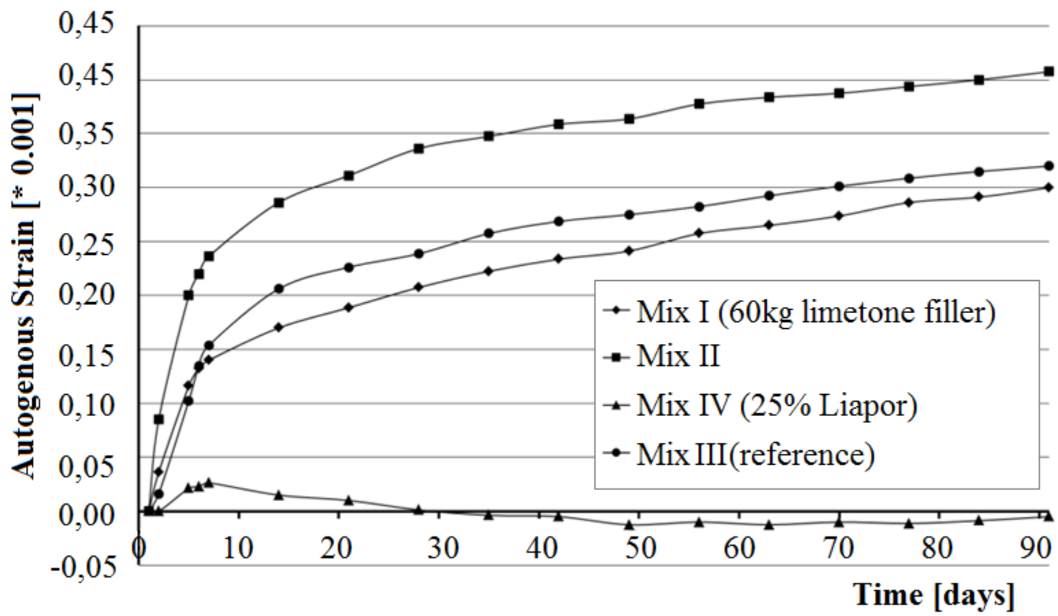

Figure 2. Autogenous deformation mixture I to IV. 20ºC. Measurements starting after 1 day (Van Breugel et.al., 2000)

5.2.3 Measured autogenous deformation and evaluation.

The autogenous deformations of the mixtures are presented in Figure 2. The measurements started after 1 day. This implies that the very first part of the autogenous deformation was not recorded. This was not considered a problem, since the aim of the test series was to quantify how autogenous deformation would affect drying shrinkage. In the practice drying shrinkage will generally not start during the first day after casting. For the purpose of this study it was appropriate, therefore, to measure only the autogenous deformation after 1 day.

The shrinkage curves of the mixtures I, II and III show that a greater part of autogenous shrinkage occurs in the first few days after mixing. But even after 28 days autogenous shrinkage still continues. From 28 to 91 days the autogenous shrinkage of the mixtures I, II and III varies from 70 to 90 μm/m. Replacing 25% of the dense aggregate by water-saturated lightweight aggregate particles was sufficient to eliminate autogenous shrinkage of this paste. Obviously, the internal curing by using saturated lightweight aggregate particles (Liapor F10, 4-8 mm) is very effective.

5.3 Autogenous shrinkage of traditional concrete mixtures C28/35 and C35/45.

The high autogenous shrinkage of mixture C55/65, much higher than expected, was reason enough to start an investigation on the autogenous shrinkage of traditional concrete mixtures with w/b ratios between 0.44 and 0.50, strength classes C35/45 and C28/35. In a preliminary study on autogenous shrinkage of concrete mixtures with w/c ≈ 0.45 Van Cappellen (2009) found that particularly at early ages the autogenous shrinkage of concrete mixtures made with blast-furnace slag cement developed faster than that of OPC-mixtures. At 200 days the difference was not very large anymore. Van Cappelle's study was continued by Mors (2011) for mixtures made with two types of aggregate, i.e. limestone and quartz. The mixture compositions are given in Table 3. Figures 3 and 4 show the autogenous shrinkage of the traditional mixtures T (0.50) and T (0.44) made with quartz aggregate and the mixtures N (0.50) and N (0.46) made with limestone aggregate. The shrinkage curves convincingly show that also mixtures with w/b in the range from 0.44 to 0.5 exhibit substantial autogenous shrinkage. More importantly, also these mixtures exhibit ongoing autogenous shrinkage at ages beyond 28 days, the age at which the concrete is generally assumed to have reached a high degree of maturity already!

Table 3. Mixture compositions of concrete mixtures C28/35 and C35/45 (Mors, 2011)

Concrete	**T (0.50)**	**T (0.44)**	**N (0.50)**	**N (0.46)**
Strength class	C28/35	C35/45	C28/35	C35/45
CEM III/B (kg/m^3)	340	340	340	360
LSP filler (kg/m^3)	--	--	20	20
Design w/c	0.50	0.44	0.50	0.46
SPL (% M/M_{cem})	0.2	0.2	0.2	0.2
Fine aggregate	Sand 0/4	Sand 0/4	Sand 0/4	Sand 0/4
Coarse aggregate	Gravel	Gravel	Limestone	Limestone
Fractions	4/8, 8/16	4/8, 8/16	6/20	6/20

T = Traditional mixture (quartz aggregate); N = Mixtures made with natural limestone as aggregate

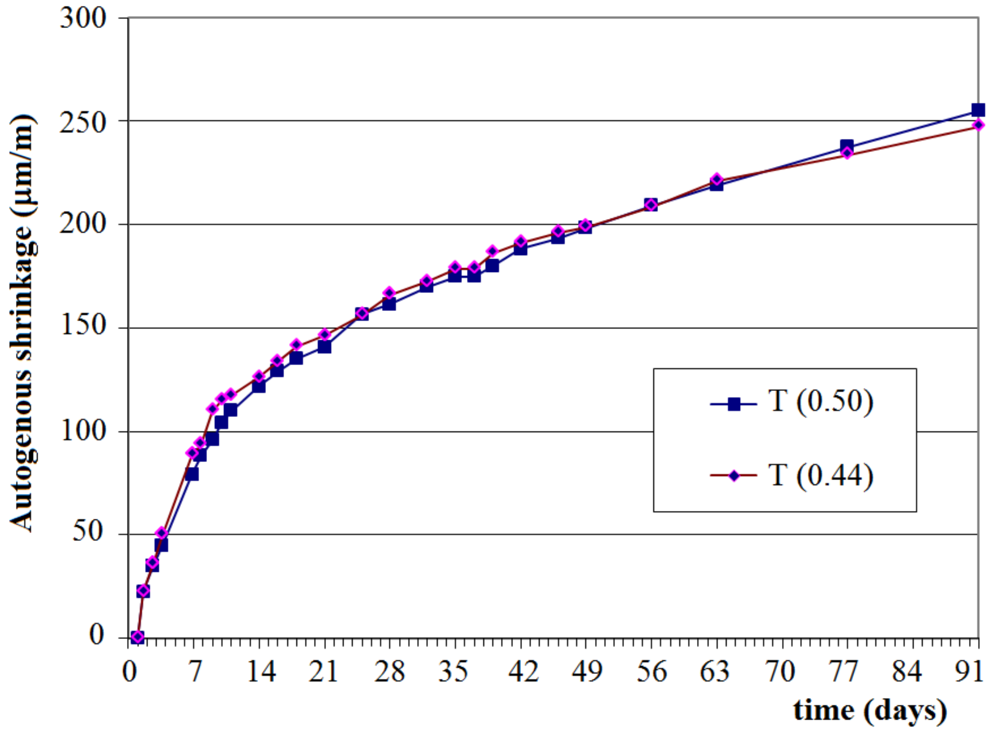

Figure 3. Autogenous shrinkage of traditional mixtures T(0.50) and T(0.44). Quartz aggregate. w/c = 0.5 and 0.44 (Mors, 2011; Van Breugel et.al., 2013)

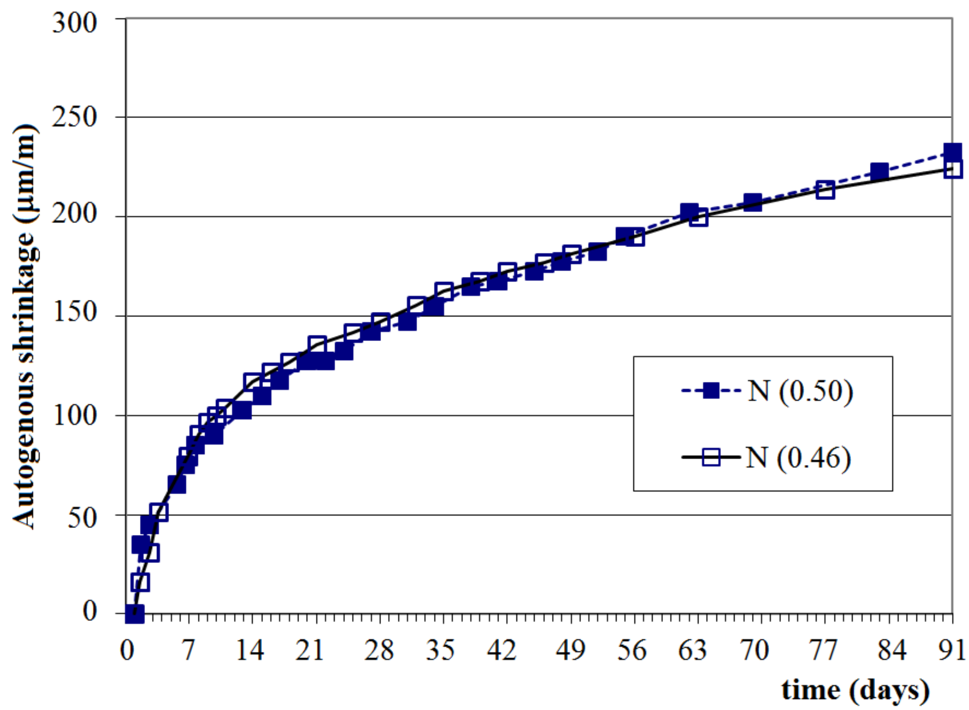

Figure 4. Autogenous shrinkage of mixtures N(0.50) and N(0.46). Limestone aggregate. w/c = 0.46 and 0.50 (after Mors, 2011; Van Breugel et.al., 2013)

5.4 Autogenous Shrinkage, Drying Shrinkage and Design Codes.

In the past experimental studies on drying shrinkage of concrete have often been carried out on 28 days old specimens and the measured shrinkage strains were mostly interpreted as drying shrinkage. From the autogenous shrinkage curves presented in the previous sections we have to conclude, however, that after 28 days autogenous shrinkage cannot be ignored, also not for mixtures with water-cement ratios higher than 0.4. For those mixtures, the contribution of autogenous shrinkage to the measured shrinkage strains in drying specimens has often been neglected. This means that in the past many drying shrinkage tests might have been misinterpreted. A substantial part of the strains measured on drying specimens should have been attributed to autogenous shrinkage. In recent updates of design codes autogenous shrinkage is now explicitly mentioned, also for traditional mixtures with w/b > 0.4. In the new EuroCode 2 and the Japanese code autogenous shrinkage is considered also for mixtures in strength classes < C55/65. For mixtures with w/b 0.44 – 0.50, however, these codes still underestimated the autogenous shrinkage, at least for the tested mixtures and cement types considered in the previous sections. Figure 5 shows autogenous shrinkage according to both the EuroCode 2 and the JSCE-Code, together with the measured autogenous shrinkage of normal strength concretes C28/35 (T(0.50)). Both the measured autogenous shrinkage and the curve after correction for small moisture loss through the sealant are presented. The autogenous shrinkage according to EuroCode 2 is presented for C28/35 and C35/45 mixtures, i.e. mixtures with strengths similar to the measured strength of the mixtures considered in this paper. In both cases the autogenous shrinkage according to EuroCode 2 is about 30% of the measured autogenous shrinkage. Underestimation of autogenous shrinkage by EuroCode 2 was also observed by Darquennes et al. (2012). The predictions with the Japanese code are closer to the measured values, but still underestimate the measured autogenous shrinkage.

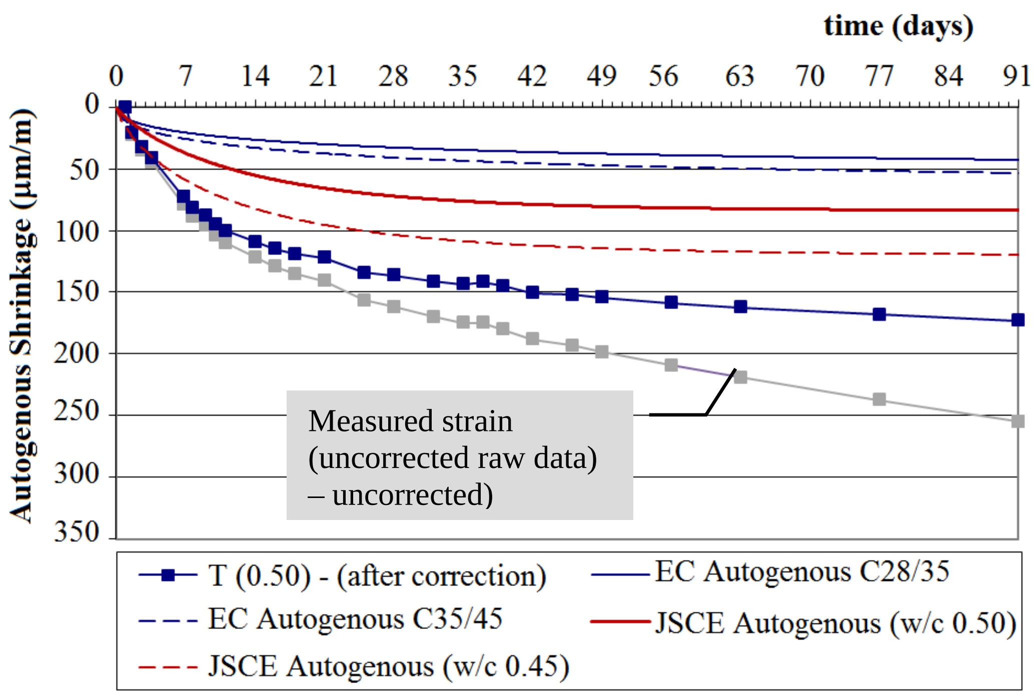

Figure 5. Comparison of measured autogenous shrinkage with predictions with the EuroCode 2 and the Japanese Code (after Mors, 2011).

5.5 Shrinkage and Ageing.

Autogenous and drying shrinkage go along with the evolution of internal stresses. In both cases the cement paste is the shrinking component. All shrinkage-induced stresses are subject to relaxation. Relaxation of stresses, however, is not 'for free'. It requires the re-structuring of smallest building blocks of matter. In other words: the material ages! As far as autogenous shrinkage strains are concerned it has been proposed that the observed long-term deformations might be creep strains following the elastic shrinkage strains exerted by the capillary forces in the pore water. Also, these creep deformations are not 'for free', but require re-structuring of the material's elementary building blocks: The material ages! For quantitative analysis of creep and relaxation Wittmann (1977) has applied the activation energy concept, which is considered the most appropriate approach for fundamental research of ageing phenomena of cement-based materials.

6. COPING WITH AGEING

6.1 Materials design.

Ageing is an inherent feature of materials. Solutions for ageing problems require, therefore, interventions at fundamental materials level. For coping with ageing problems, two approaches are conceivable, i.e. the preventive approach and the reactive approach.

In the *preventive* approach the focus is on designing homogenous materials with as few as possible internal gradients, stress concentrations and interfaces. For heterogeneous materials, like concrete, this is a big challenge. When going through the subsequent length scales, from (sub-) nano to meso level, concrete behaves as a complex system. To state it differently, concrete is a 'product of the mind' (McCarter, 2009), of which the properties are determined by the properties of the individual components and of the interfaces between them. Some of these components – in fact all! - change with time, and so do the properties of the interfaces. This makes heterogeneous materials susceptible to ageing.

In the *reactive* approach the heterogeneity of the material, and hence the internal concentrations of stresses and strains and the occurrence of internal damage and ageing, are considered a matter of fact. If ageing is unavoidable indeed, *self-healing* could be a solution for ageing problems. When dealing with concrete the presence of still unhydrated cement particles provides an inherent self-healing capacity. In this respect concrete made with a coarse cement is considered favourable to concrete made with a fine cement. Mehta's (2001) observation that old bridge decks, built with coarse cement, have performed better than the younger ones built with finer cements, could be explained, at least in part, by the role of self-healing in the older structures. The modern trend to, firstly, use finer cement to speed up the rate of strength gain and, secondly, reduce the amount of cement to reduce the CO_2 footprint of concrete, may work out negative on the material's resistance against ageing! In these cases, a comprehensive life cycle analysis is needed for weighing all the pros and cons of modern trends in concrete mix design.

6.2 Ageing and design codes.

For designing and realizing concrete structures design codes are indispensable. From the numerous buildings and fascinating construction works realised in the past a high degree of maturity of these codes can be inferred. In section 5.4 we have seen, however, that currently used prescriptive codes fall short in describing long-term performance, i.e. shrinkage, of concrete structures. In this respect, it is interesting to reflect on the recent tendency to switch from prescriptive codes to performance-based codes. The question is whether it is to be expected that with this switch ageing issues will be considered more appropriately and will become part of an integral design approach for concrete structures. Strictly speaking the change from prescriptive to performance-based codes is a return to the origin of the building profession. In the ancient past

the whole building process was in the hands one person: the builder. The builder had the integral responsibility to meet all the safety and functional criteria set by the owner. How the builder managed to meet the owner's criteria was not prescribed in detail. This was all considered the builder's expertise and responsibility. In his classical book on building technology Vitruvius (85-20 BC) stated that ideally the whole building process should be in the hands of one person. When Vitruvius wrote his book, a few years BC, he noticed already that this ideal situation was no longer tenable. The building process became too complicated and one single person could not be an expert in all areas of the building process. Gradually the builder had to share his responsibility with others. This situation started the emerge of certificates etc., and, later on, prescriptive codes. The user of these documents could be held responsible for a correct interpretation of and compliance with the codes, but not for the content of the codes.
Prescriptive codes can be judged as the ultimate consequence of a process of increasing fragmentation of the building process and, more importantly, of the vision that everything, including quality, is engineerable. The huge sustainability problems we face today, however, illustrates that this vision has lost most of its convincing power. Prescriptive codes, even the most detailed ones, are *necessary*, but *insufficient* for guaranteeing quality and/or sustainability. Prescriptive codes deal with materials properties with the primary goal to provide the designer with data needed for designing safe structures. Any change of performance of the materials with time is considered a time dependent property without addressing the cause of these changes.
With performance-based codes the building process has been given back to the builder, including the challenge to accomplish (long-term) quality criteria and sustainability goals. The builder's freedom to decide how to meet these criteria and goals may stimulate the builder to invest in fundamental research of traditional and new building materials and in innovative design concepts. Furthermore, performance-based codes, in combination with new DBFM (Design-Built-Finance-Maintenance) contracts, will also force the builder to focus on both the short-term and long-term performance, i.e. ageing, of materials and structures. For that purpose, the builder will need reliable predictive models, including models for quantifying the rate of ageing processes and the consequences thereof.

7. REQUIRED INVESTMENT FOR GENERATING SAVINGS

In section 2 it has been explained that ageing of the nation's fixed capital goods is a huge financial burden for the society. A way to reduce this burden is by reducing the maintenance costs and extending the life time of our infrastructure. This will result in savings in annual replacement costs of obsolete structures. But for realizing these savings we first must invest! Potential savings justify, and require, investments in ageing research. Some key figures may help us to get an indicative picture of the required investment for realizing a certain level of savings. In section 2 the global value of the infrastructure stock has been estimated at € 37 trillion. Let us assume an average lifetime of these infrastructure assets of 50 years. Each year € 740 billion has then to be spent on replacement of obsolete assets. Let us further assume that through dedicated research the average lifetime can be increased by 10%, i.e. from 50 to 55 years. The yearly replacement costs would then decrease to from € 740 to € 670 billion. This is a reduction of € 70 billion per year. Let us assume that for saving these € 70 billion we must invest 20% of this amount in research, i.e. € 14 billion per year. Let us further assume that 50% of the required research money, i.e. € 7 billion, must be spent on management-oriented research and the other 50% on science-oriented research on materials and structures. A part of this science-oriented research must be spent on ageing research. A reasonable, though conservative assumption is that 10% of science-oriented research, i.e. € 0.7 billion per year, should be spent on fundamental ageing research. This € 0.7 billion is only 1% of the targeted savings. Schematically this is shown in figure 6. By varying the assumptions in this exercise other values for required investments are obtained, but do not change the order of magnitude of these figures.

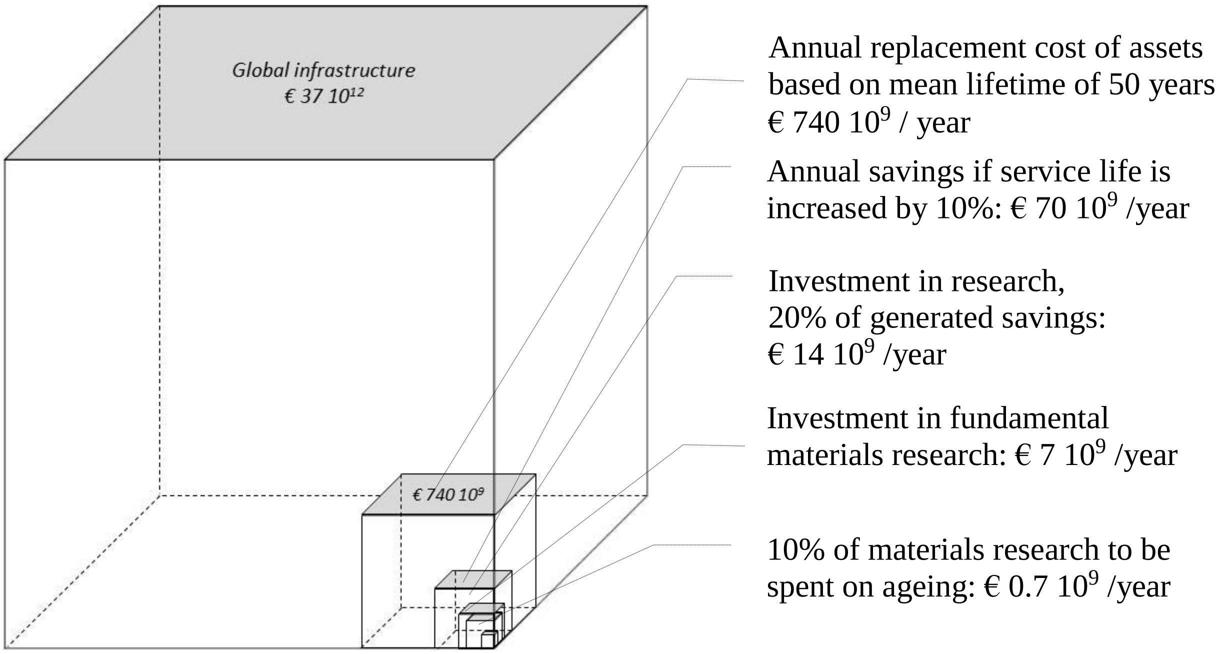

Figure 6. Schematic presentation of required investment in ageing research for realizing an extension of the mean service life of infrastructure of 10%. (interest / inflation not considered). Estimated average lifetime 50 years.

8. CONCLUDING REMARKS

A nation's infrastructure makes out about 50% if its national wealth. This huge share of our national wealth, however, is ageing! With the existence, growth, maintenance and replacement of ageing infrastructure a huge responsibility comes to all actors involved in planning, design, building and operating our assets. It is a matter of *responsible stewardship* to mitigate the environmental impact that comes along with realizing and operating our infrastructure.

Fundamental research on ageing is recommended to improve the tools for accurate and reliable predictions of the long-term performance of our ageing infrastructure. The results of experimental research on autogenous shrinkage of traditional and modern, innovative cement-based materials illustrate the need for more research to better understand the cause of autogenous shrinkage, as well as the (sometimes unexpected) possibilities to mitigate autogenous shrinkage, for example by using low-tech waste products, like rice husk ash (Tuan, 2011). Mitigating shrinkage implies mitigating shrinkage-induced stresses and hence reducing the rate of ageing.

Performance-based design codes, in combination with new contracts in which the builder is made responsible for the long-term performance and operation of their structures, generate a strong need of knowledge of ageing phenomena in materials and structures. In this way performance-based codes may stimulate the search for innovative solutions.

Like many other industries, also the building industry is under pressure. Structures must be realised faster, but with lower environmental impact. Any product, however, realised under pressure, irrespective of what kind of pressure, has an inherent tendency to age. To cope with the risk of increasing ageing rates, in-depth knowledge of the performance of materials and structures with elapse of time is needed.

An increase of the average service life of our infrastructure by 10% would save tens of billions of euros each year. The required investments to realise these savings are estimated at 20% of these savings. Half of this amount is assumed to be needed for research on materials and structures, of which 10% has been assumed to be needed for fundamental research on ageing. Setting such targets for savings is not only challenging and a stimulus for research and innovation. The figures also illustrate that caring for our infrastructure will finally pay off.

9. ACKNOWLEDGEMENTS

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Development of pseudo interface element for modelling of reinforced brick masonry

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ABSTRACT
Strength of reinforced masonry is influenced by interfaces between brick, mortar and reinforcement. Experimental protocol has been defined to characterise the behaviour of reinforced brick masonry joint, with reinforcement steel embedded in cement mortar 1:6. This is applicable for low-strength, low-stiffness brick masonry found. Experimental investigations show that bond between masonry and steel is not perfect. Considering critical bond mechanisms, an attempt is made to put-forth a novel approach for development of a pseudo interface element representing three different materials (viz. brick, mortar and reinforcement) and two interfaces (reinforcement-mortar (RM) interface and brick-mortar (BM) interface). Principles of classical Reinforced Concrete (RC) design can therefore be directly applied to reinforced masonry with the introduction of the proposed pseudo interface element.
Keywords: reinforced masonry joint; interface element; masonry reinforcement bond behavior; pseudo interface material; stiffness of interface elements.

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Desarrollo de un pseudo-elemento de interfaz para el modelado de mampostería de ladrillo reforzado

RESUMEN

La resistencia de la mampostería reforzada está influenciada por las interfaces entre el ladrillo, el mortero y el refuerzo. Se ha definido un protocolo experimental para caracterizar el comportamiento de la junta de mampostería de ladrillo reforzado, con acero de refuerzo incrustado en mortero de cemento 1: 6. Esto es aplicable para la albañilería con ladrillos de baja resistencia y baja rigidez encontrada. Las investigaciones experimentales demuestran que el vínculo entre la mampostería y el acero no es perfecto. Teniendo en cuenta los mecanismos de enlace críticos, se intenta presentar un nuevo enfoque para el desarrollo de un elemento de pseudo-interfaz que represente tres materiales diferentes (ladrillo, mortero y refuerzo) y dos interfaces (de refuerzo y mortero (RM) y de mortero (BM)). Por lo tanto, los principios del diseño de concreto armado (RC) clásico pueden aplicarse directamente a la mampostería reforzada con la introducción del pseudo-elemento de interfaz propuesto.

Palabras clave: articulación de mampostería reforzada; elemento de interfaz; comportamiento de enlace de refuerzo de mampostería; pseudo-material de interfaz; rigidez de los elementos de la interfaz.

Desenvolvimento de elemento de pseudo interface para modelagem de alvenaria de tijolo armado

RESUMO

A resistência da alvenaria reforçada é influenciada pelas interfaces entre tijolo, argamassa e armadura. O protocolo experimental foi definido para caracterizar o comportamento de juntas de alvenaria de tijolo armado, com aço embutido em argamassa de cimento 1:6. Isto é aplicável para baixa resistência, com tijolo de baixa rigidez. Investigações experimentais mostram que a ligação entre a alvenaria e o aço não é perfeita. Considerando os mecanismos de ligação críticos, é feita uma tentativa de apresentar uma nova abordagem para o desenvolvimento de um elemento de pseudo interface representando três materiais diferentes (tijolo vizinho, argamassa e armadura) e duas interfaces (interface argamassa-armadura (RM) e interface tijolo-argamassa (BM)). Os princípios de projeto clássicos de concreto armado (RC) podem, portanto, ser diretamente aplicados à alvenaria armada com a introdução do elemento de pseudo interface proposto.

Palavras chave: junta de alvenaria armada; elemento de interface; comportamento de ligação de reforço de alvenaria; material de pseudo interface; rigidez dos elementos de interface.

1. INTRODUCTION

Masonry is a brittle construction material that has been used for a very long time around the world and is still being used. Over the period, masonry is used as vertical load carrying elements due to excellent performance in compression. Limited tensile capacity of masonry is generally overcome by using arches, vaults, etc. over opening. These arches, vaults convert flexural tension into compression due to their geometry. In comparison, concrete is also a brittle material with limited tensile capacity and generally this limitation is overcome by the introduction of reinforcing steel or by pre-stressing. Similar use of reinforcement in masonry construction is not new, but is uncommon in India. Reinforcement can be introduced in masonry elements in several ways.

The most common method is to place reinforcing bars in the bed joints. Structural members built in this way can be used to resist flexural forces (loads), in the form of beam. Most of the codes

available for reinforced masonry are based on principles and assumptions of Reinforced Concrete (RC) design. Main assumption of classical RC design is that, the tensile force is resisted by reinforcement alone and bond between reinforcement and concrete is nearly perfect.
Literature on brick masonry reveals that in western countries bricks are more stiff and stronger than mortar used. Compressive strength of such bricks may be in the range of 15-150 MPa and elasticity modulus in the range of 3500-35000 MPa. Whereas in India, bricks have relatively less compressive strength (3-20 MPa), and elastic modulus (300-15000 MPa). Also, the commonly used cement mortar (1:6) generally have elasticity modulus 10 to 15 times higher than that of bricks (Matthana, 1996), (Sarangapani et al, 2005), (Raghunath et. al., 1998) and (Gumaste et al, 2004). Laurenco 1994, has enlisted various models to predict behaviour of unreinforfced masonry. Laurenco recommended Coloumb friction model with compression cap for interface between mortar and brick. Globally, classical RC theory is used for modelling reinforced masonry (Narendra Taly, 2010).
Typically, reinforced brick masonry in flexure is achieved by inserting reinforcement in bed joint at certain depth. The joint assemblage in reinforced masonry comprises of five elements viz. (i) reinforcement, (ii) reinforcement-mortar interface, (iii) mortar, (iv) mortar-brick interface and (v) brick (units). These are shown schematically in Figure 1.

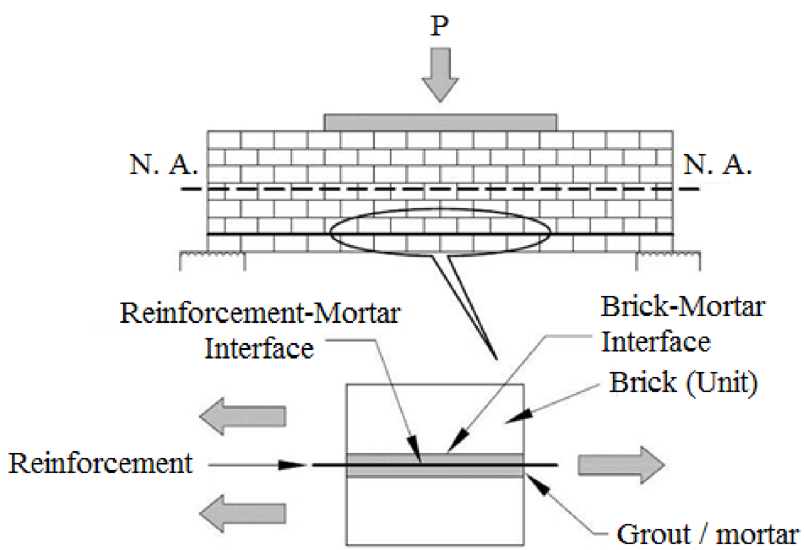

Figure 1. Typical reinforced masonry beam and reinforced masonry joint details.

Experimental investigations performed on reinforced brick masonry (Shashank Mehendale et. al. 2016) show that, bond between reinforcement and brick masonry is not perfect. Different shear deformations are being observed due to variation in shear properties of individual elements and interfaces between them; leading to loss in strain and less force is developed in the reinforcement as compared to a scenario of perfect bond. Thus, the contribution of reinforcement in reinforced masonry beam is likely to be lesser than that of RC beam. It is observed that weakest links in reinforced masonry are the interfaces between brick, mortar and reinforcement. In reinforced masonry design, use of classical assumptions of design of RC may lead to over reliance on reinforcement. The need for novel approach for design of reinforced masonry beam is felt for low strength type bricks and mortars used in the study.

Considering the importance of interfaces, detailed investigation of individual elements of reinforced masonry joint in similar test environment was carried out. Based on the results of experimental work on individual elements and assemblage, an attempt is made to develop a pseudo interface element. The present work aims at using experimental observations of individual elements and merging the same into a pseudo interface by suitably capturing the contribution of each of the elements in the assemblage. The proposed pseudo interface element can be lumped with masonry, thereby improving predictions about contribution of reinforcement. The purpose of the present research is to study and develop design protocol, which will help to achieve the optimum utilisation of material and efficient joint.

2. EXPERIMENTAL INVESTIGATION

Behaviour of reinforced masonry joint is investigated using experimental protocol presented in this study. Pull-out test is widely used as an effective means for the characterization of the bond behaviour between internally bonded reinforcements and masonry. An indigenously developed test set-up shown in Figure 2 (a) and (b) are used to study the behaviour of reinforced masonry assemblage using facilities available in VJTI laboratory.
Locally available country moulded bricks with cement mortar 1:6 has been used for preparation of samples. Water cement used in mortar was based on flow test. 8mm diameter HYSD steel bar were placed at centre of 20 mm thick mortar layer in assemblage. A counter weight of 2 bricks was maintained over each sample for 4 days to ensure proper bonding between mortar and bricks. The samples were cured for 14 days. Expected over burden pressure in in-situ condition is simulated in tests by applying confining pressure to samples. Over burden pressure generally found to be around 0.5 N/mm^2 (Laurenco 1994) was used in present experimental investigation. Pull-out force was applied using strain controlled device and deformation (displacements) response was recorded.
Figure 3 shows the plot of pull-out force vs. Displacement of the reinforcement. It is observed that pull-out force of varies with displacement of reinforcement bar almost linearly up to peak value of force, thereafter, softening is observed as displacement increases. It can be noticed that some residual buffer capacity exists due to skin friction effect. It is observed from experiments that the residual buffer capacity is a function of confining pressure. As the properties of the masonry unit is not consistent even in a single lot, 20 numbers of tests were planned to get representative and reliable results.

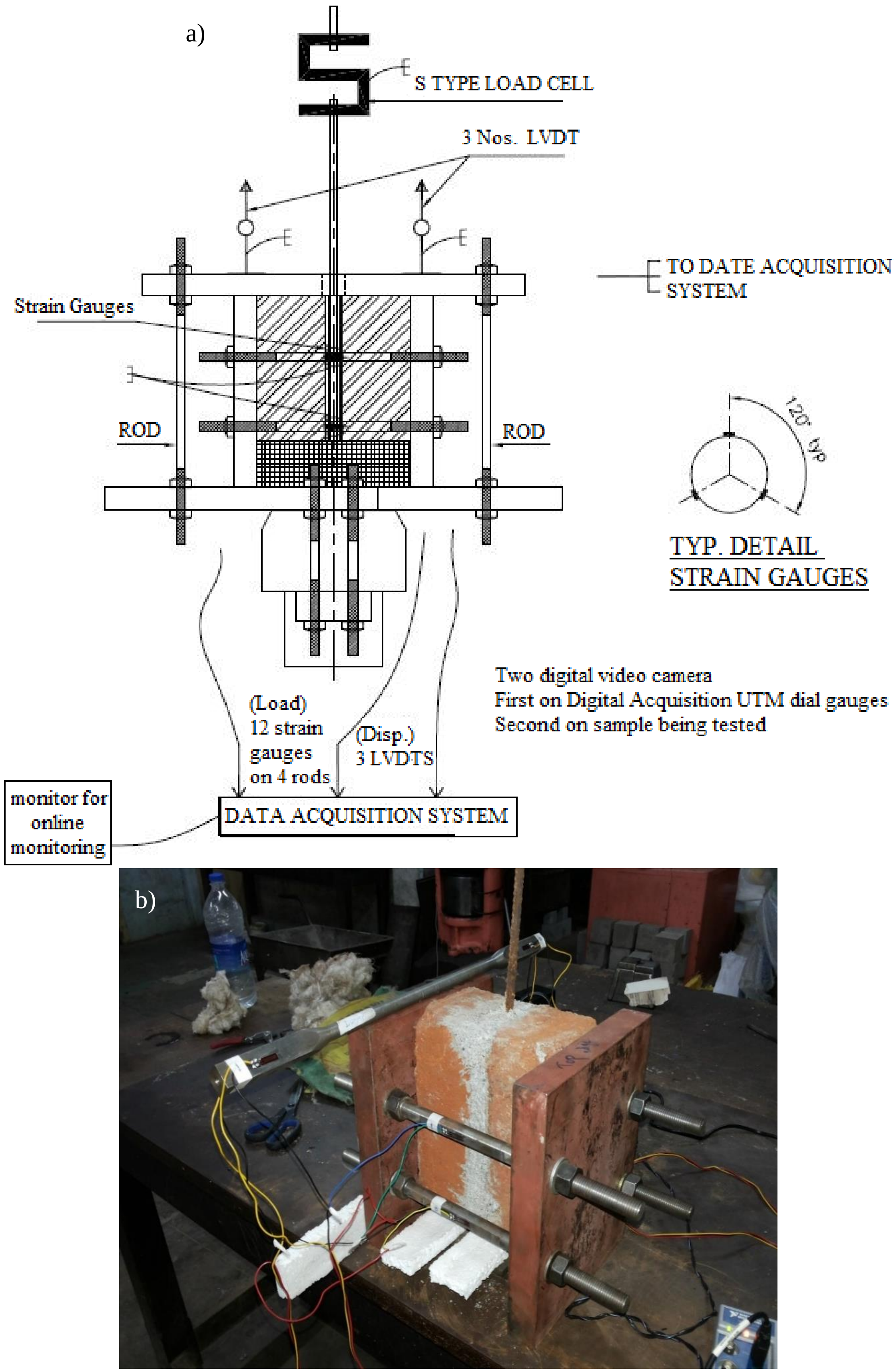

Figure 2. (a) Sample held in position (b) Pull-out Test set-up

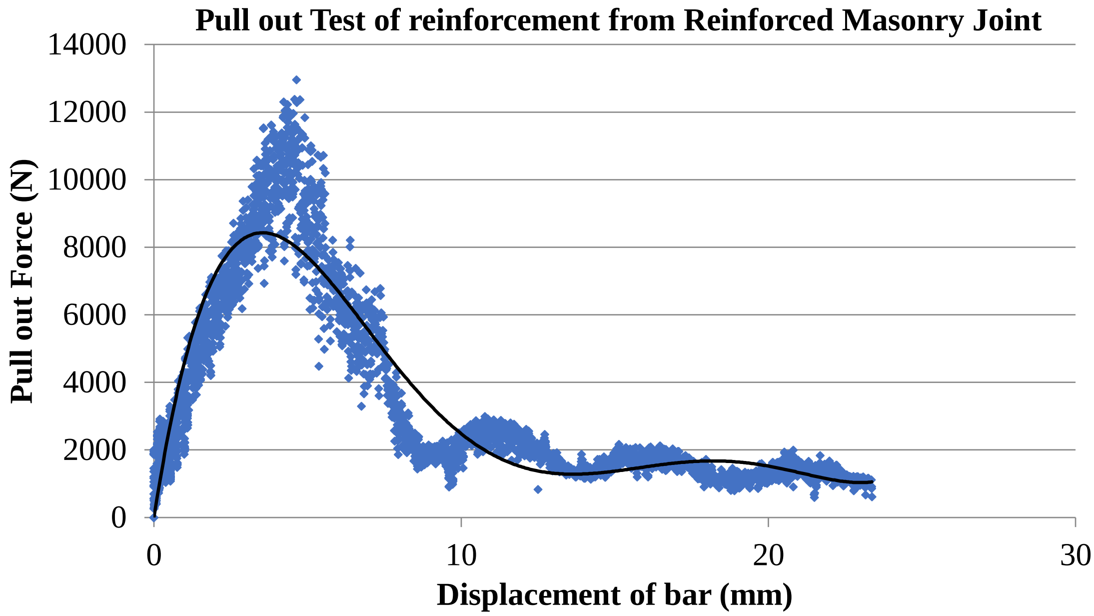

Figure 3. Results of Pull-out Test on assemblage.

The pull-out capacity of bar embedded in assemblage and associated stiffness depends on the complex interaction between individual elements viz. brick, mortar, reinforcement and interface between brick and mortar and interface between reinforcement and mortar. This behavior is found to be different than that in an RC member. It is observed that strain in extreme fiber brick masonry is not fully transferred to reinforcement due to shear slippage of quassi-brittle material viz. brick and mortar.
To study the various parameters affecting pull-out capacity and stiffness; based on the available literature (Laurenco, 1994), various test set-ups have been fabricated and experiments were performed to determine the properties of units, mortar, reinforcement and interfaces used. Details of set-ups and experimental protocol is briefly described in this study. Table 1 shows the properties of basic elements used in study, which represents the reinforced masonry joint.

Table 1. Properties of Materials used in study

Tests	Brick	Mortar (1:6)	Reinforcement (8mm dia.)
Compressive Strength (MPa) (Number of Specimens)	3.88 (8)	8.32 (06)	-
Flexure Strength (MPa) (Number of Specimens) Note: loaded along depth	0.98 (6)	2.42 (06)	-
$E_{initial\ tangent}$ (MPa)	142.2	15401.6	2 X 10^5
Tensile Strength (MPa)	-	0.96	415

A. Tensile Test on Reinforcement

Tensile test on reinforcement was carried out using procedure prescribed in IS 1786 (2008). Axial stiffness of reinforcement is a contributing parameter.

B. Pull-out Test of Reinforcement from Mortar alone

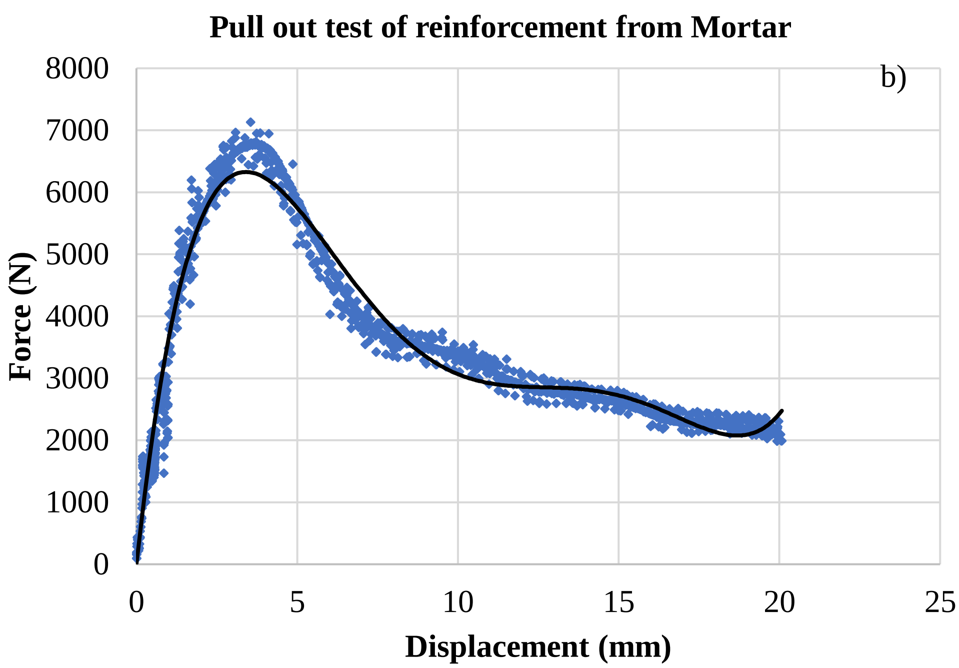

Figure 4. (a) Specimen (b) Results of Pull-out Test of R/f from Mortar

Pull-out test of reinforcement from mortar was carried out using the above referred test set-up. Sample size used was same as that of the assemblage (160mm x 200mm x 90mm), with reinforcement placed at center. This test is used to determine reinforcement mortar interface properties. Confinement pressure of 0.5 N/mm^2 is applied using tension bolts. Figure 4 shows the plot of pull-out force in reinforcement vs. displacement.

C. Mortar double shear test

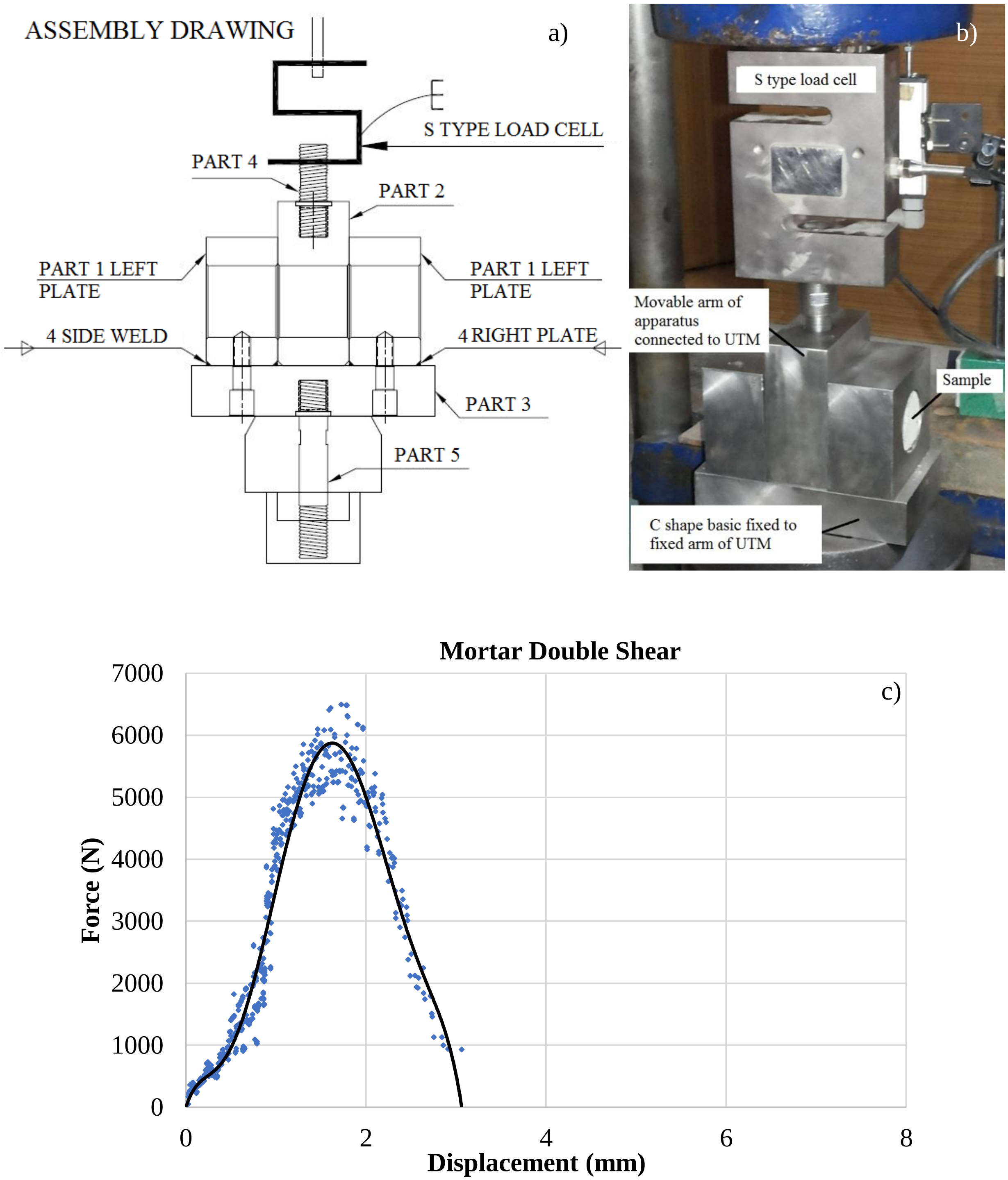

Figure 5. (a) Schematic Mortar Shear Test Set up, (b) actual photograph (c) Results

Mortar shear contribution is established using indigenouelsy developed Mortar Double Shear Apparatus. This apparatus is based on concept of double shear, often used in soil mechanics. Sample size of 50mm diameter mortar cylinder with 150mm length was adopted. Apparatus, shown in Figure 5, consist of 2 main elements, lower element (C shape) consists of part 1 (left and right plate) and part 3 are attached to fixed arm of UTM. Top element labelled as part 2 is attached to movable arm of UTM thru load cell.

D. Brick-Mortar Interface Shear Test

Brick-Mortar interface shear test has been carried out with confinement pressure applied, using the arrangement shown in figure 6. Epoxy was used to ensure perfect bond between sample and apparatus. Shear contribution of brick-mortar interface is calculated using shear test apparatus suggested by P B Launreco, (1994) and Van der Plujim, (1992 & 1993).

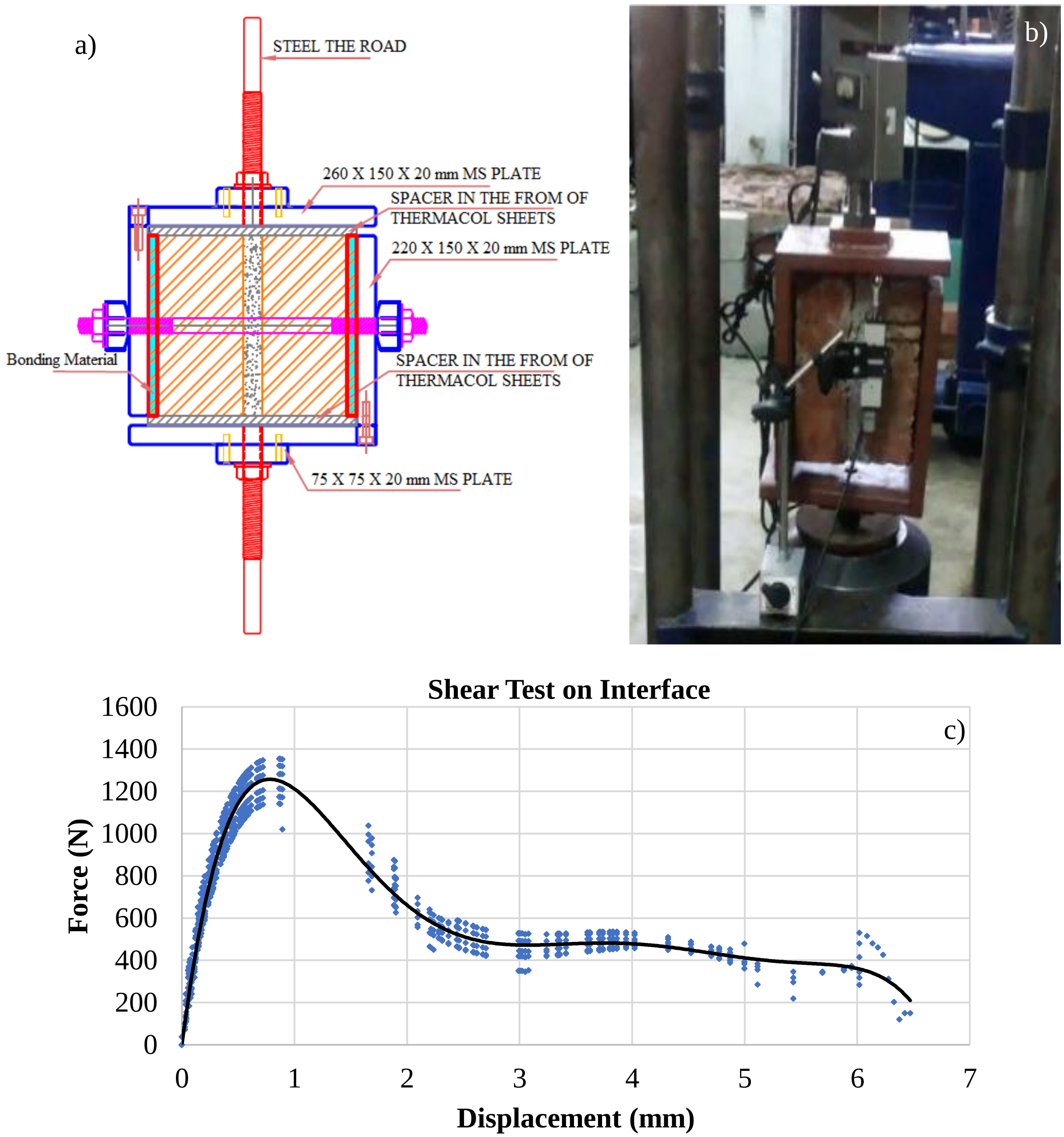

Figure 6. Shear Test on Brick Mortar Interface (a) Schematic Test set-up (b) Actual photograph (c) Results

E. Brick Shear Test

Brick shear contribution is worked out from direct shear test concept commonly used in geotechnical engineering. Confinement pressure plays an important role in shear strength evaluation. In all the above mentioned experiments, a confinement pressure 0.5 N/mm^2 has been used brick samples of 50 mm x 50 mm cross-section were cut carefully from brick units. The test set-up, failure pattern observed and test results are shown in figure 7 (a), (b) and (c) respectively.

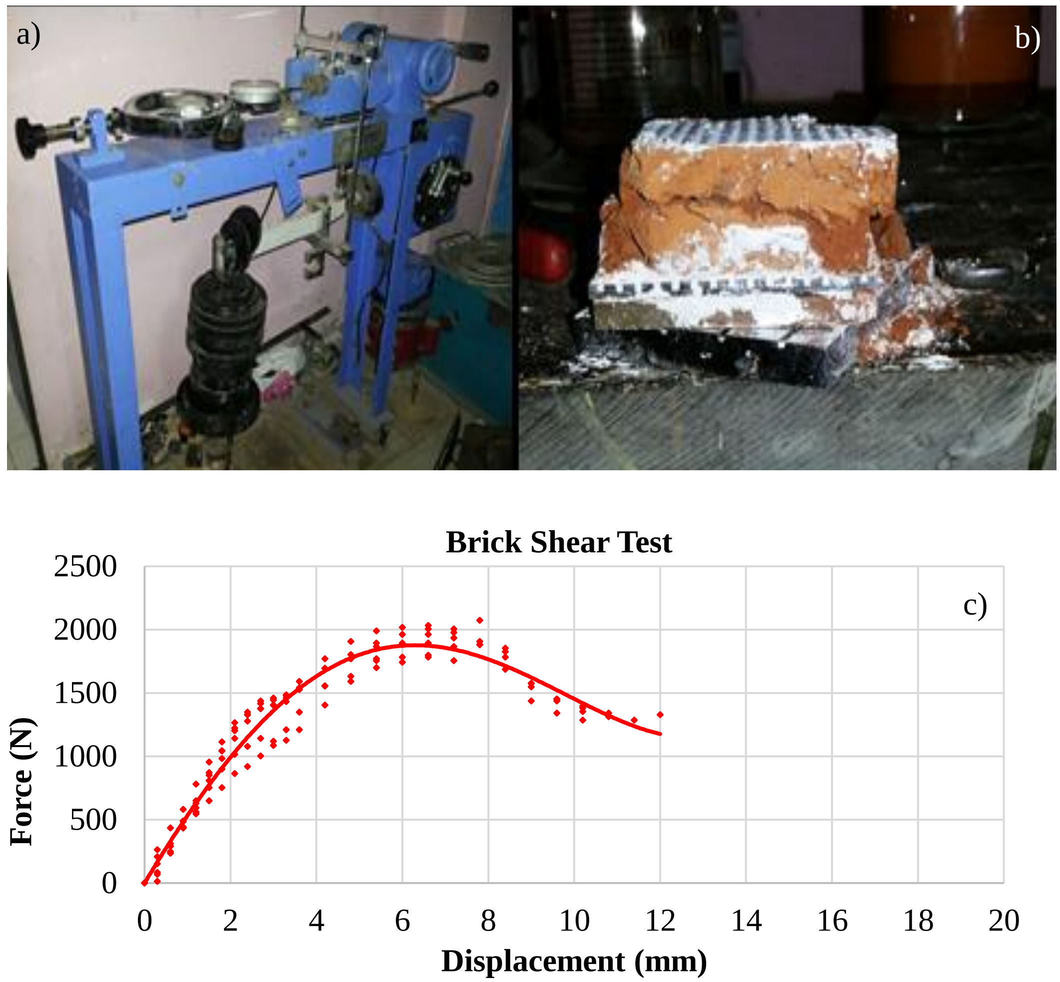

Figure 7. (a) Brick shear test test set-up, (b) Typical shear failure observed and (c) Brick Shear Test Results

The large deformations observed are not relevant to design process as structure would have failed by the time large deformations take place. Thus, the attempt of designer is to keep deformations within control and as minimum as possible. Within such controlled and limited range of deformations behavior of individual elements and interfaces can be assumed to linearly elastic. Thus, it can be idealized as discrete springs.

An idealization using spring analogy has been put forth to simplify the complexity due to contribution of five elements of the assemblage. Pull-out force remains constant across all the elements and deformation is a function of the resistance offered by reinforcement, reinforcement-mortar interface, mortar, mortar-brick interface and brick resistance. Each contributing parameter can be idealized as discrete springs and the entire assemblage can be idealized as system of springs connected in series. Thus, the effective stiffness of assemblage is a contribution of stiffness of reinforcement, shear stiffness of RM bond, shear stiffness of mortar, shear stiffness of BM bond and shear stiffness of brick. A schematic representation of the above-mentioned idealization is represented in Figure 8.

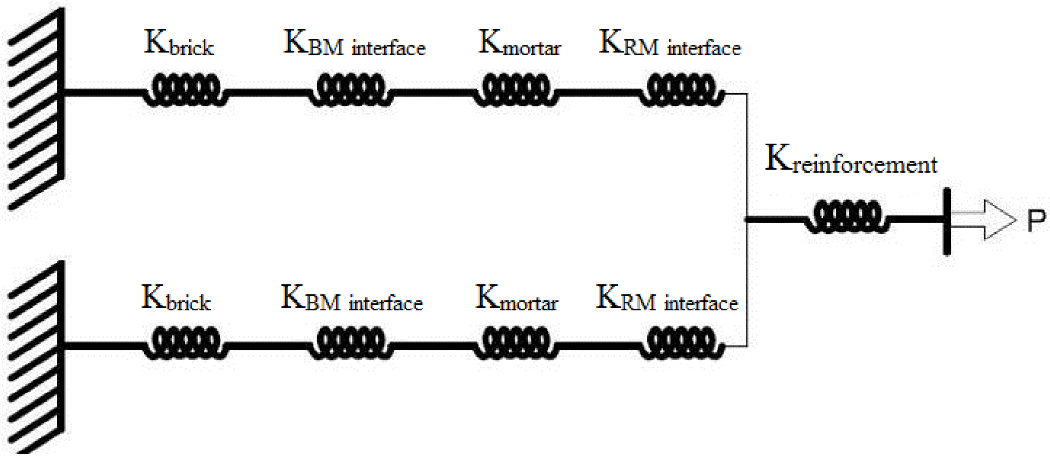

Figure 8. Spring Analogy

3. EXPERIMENTAL FINDINGS AND INFERENCE

All tests were performed using displacement controlled machine at VJTI, Structural Engineering laboratory. A plot of Force vs. Displacement graphs were obtained for each test. Stiffness values of each contributing element have been calculated and tabulated in Table 2.
It is observed that there is good correlation between the experimental stiffness for the said joint and the results obtained using analytical model using spring idealization. Spring formulas for springs in parallel and series in combination for the above assemblage of springs shown in Figure 8 can be represented as follows using Equation (1) and Equation (2);

$$\frac{1}{k_{Eq.\text{pseudomaterial}}} = \frac{1}{k} + \frac{1}{k_{reinforcement}} \tag{1}$$

Where, k is equivalent spring constant for springs in parallel.

$$\frac{1}{k} = \frac{2}{\dfrac{1}{k_{RM\,interface}} + \dfrac{1}{k_{mortar}} + \dfrac{1}{k_{BM\,interface}} + \dfrac{1}{k_{brick}}} \tag{2}$$

Equation (1) and Equation (2) are valid within elastic bounds of each springs, i. e.

$$\text{P} \le \; P_R^L,\; P_{RM}^L,\; P_M^L,\; P_{BM}^L,\; P_B^L \tag{3}$$

Where,

P_R^L (limiting elastic load on reinforcement spring)
= Limiting elastic stress of Reinforcement x C/s area of reinforcement

P_{RM}^L (limiting elastic load on RM interface spring)
= Limiting elastic stress of spring equivalent to RM interface x C/s area of RM interface

P_M^L (limiting elastic load on mortar spring)
= Limiting shear stress of spring equivalent to Mortar x C/s area of mortar in shear

P_{BM}^L (limiting elastic load on BM interface spring)
= Limiting stress of spring equivalent to BM interface x C/s area of BM interface

P_B^L (limiting elastic load on brick spring)
= Limiting shear stress of spring equivalent to Brick x C/s area of brick in shear

Breach of any of these bounds i.e. failure of any of above listed idealized springs shall predict the corresponding failure mode e.g. failure of RM spring will indicate failure of interface between reinforcement and mortar.

Table 2. Stiffnesses Matrix (Experimental and formula based).

Experimental Element stiffness	**Initial tangent Stiffness**
$K_{reinforcement}$	25120 N/mm
$K_{RM\ interface}$	3521.1 N/mm
K_{Mortar}	2857.1 N/mm
$K_{BM\ interface}$	6666 N/mm
K_{Brick}	714 N/mm
$K_{Eq.\ pseudo\ material}$ (from Formula)	883.77 N/mm
$K_{Eq.\ pseudo\ material}$ (Pull-out test)	981 N/mm

This correlation using spring analogy can be used to model pseudo interface element with effective stiffness equivalent to that of reinforced masonry joint.

3.1 Relationship to Elasticity

Spring analogy idealization presented above forms the basic approach for development of a novel formulation for modelling pseudo interface material for reinforced brick masonry joint. The joint in flexural formulation resists tension where the masonry takes the compression. Thus, the joint can be considered to be subject to tension alone. The stiffness k, of a body is a measure of the resistance offered by an elastic body to deformation and is thus ratio of force applied to the deformation produced. In mechanics, the Elastic modulus (Young's Modulus) is an intrinsic property of material that is computed as the ratio of stress to strain, i.e.

$$E = \frac{\sigma}{\varepsilon} \tag{4}$$

substituting values for σ and ε, $E = \frac{F/A}{\Delta/l}$ i.e.

$$E = \frac{Fl}{A\Delta} \tag{5}$$

From above assumptions and rearranging the terms, above equation can be restated as

$$E = \left(\frac{F}{\Delta}\right)\left(\frac{L}{A}\right) \tag{6}$$

i.e. $$E = K\frac{l}{A} \tag{7}$$

Thus, tension modulus of elasticity for pseudo material can be written in terms of reinforced masonry joint stiffness k using Equation (4) as follows,

$$E_{Eq.pseudomaterial} = \frac{L}{A} k_{Eq.pseudomaterial} \tag{8}$$

Substituting experimental stiffness values of pseudo interface element, the modulus of elasticity of pseudo interface element will be, (for unit length reinforced masonry beam with unit cross sectional area), $E_{eq.pseudomaterial} = 883.77 MPa$.

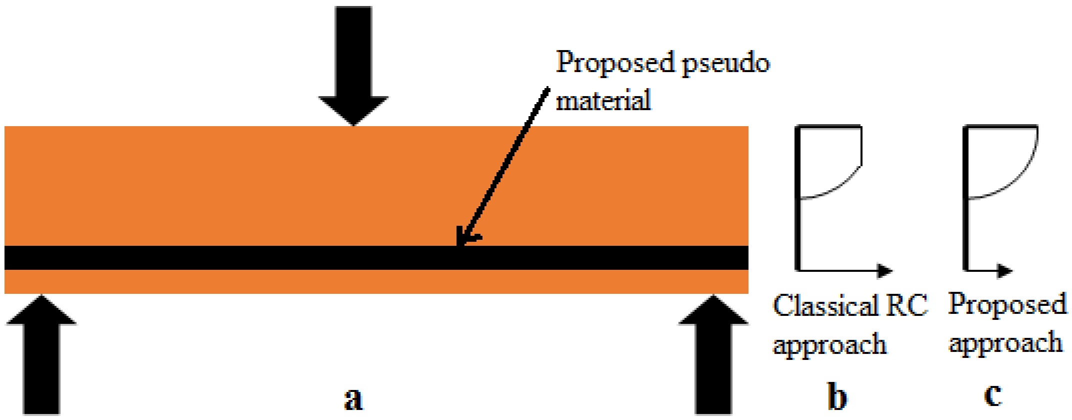

Figure 9. Typical reinforced masonry beam with stress relationship b) Stress diagram as per RC design assumption, c) Stress diagram as per study with discounted tensile force and masonry stress block.

Comparison of stress block for reinforced masonry using classical RC design approach (figure 9 b) and proposed formulation for modelling interface of reinforced masonry (figure 9 c) is shown in figure 9. In classical design approach reinforcement yields first and allowed to take load upto its yielding capacity. Where as in this approach, the pseudo interface element yields when one of the interface boundary conditions is breached. This results in relatively lesser contribution by reinforcement as compared to classical RC design. Thus, a reinforced masonry beam would require larger depths and profiled reinforcement grouted into masonry so as not to breach the interface boundary condition and this approach is leads to optimum solution.

4. CONCLUSION

Investigations performed on reinforced masonry with focus on the materials viz. unit, mortar and reinforcement and interfaces indicates following features.

i) Bond between reinforcement bar and masonry is not perfect.
ii) Due to relative shear deformations loss in strain occurs and thus less force is developed in the reinforcement bar.
iii) Relative shear deformations are observed due to different shear properties of individual elements and interfaces between them.
iv) Developed pseudo interface element predicts likely failure modes.

Behavior of reinforced masonry is different from RC, hence assumptions of classical RC design cannot be directly used for considered masonry units. This difference would be more for weak bricks and weak mortars. Considering the complexity of reinforced masonry joint, this study has presented an approach to develop a pseudo interface element representing 5 different elements of a reinforced masonry joint. This pseudo interface element would help in design and modelling of reinforced masonry in flexure. Though, tests are carried out on particular type of brick (unit), mortar/ grout and reinforcement, the experimental protocol and proposed approach for development of pseudo interface element in this study is robust enough and can be used to other types of units, mortars and reinforcing materials. Same approach might be helpful in similar type of conditions to simplify interface complexities. The developed pseudo interface element would help engineers to arrive at the most suitable and economical reinforced masonry solution.

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Sustainability evaluation of different techniques for concrete mixing based on quality control

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ABSTRACT
In this study emergy analysis, an environmental valuation method was applied to concrete mixing with the purpose of evaluating its dependence on non-renewable natural resources. Three concrete mixing techniques, industrialized, semi-industrialized and manual, were evaluated based on quality control. The quantity of environmental resources used in production was measured in terms of equivalent solar energy. The resulting transformities were compared to show that emergy analysis is sensible to local context and the limits of the reference system. The results obtained show that concrete mixing is highly dependent on external resources. Semi-industrialized concrete was found to be the most sustainable.
Keywords: Emergy analysis; environmental accounting; sustainability; transformity; concrete.

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Evaluación de la sustentabilidad de diferentes técnicas de producción de concreto hidráulico basado en su control de calidad

RESUMEN

En este trabajo, se aplicó un método de valoración ambiental en la producción concreto con el fin de evaluar su dependencia de los recursos naturales no renovables. Tres técnicas de producción basadas en su control de calidad: industrializada, semi-industrializada y manual; se evalualuaron mediante eMergía. Esto se realizó para medir la cantidad de uso de los recursos del medio ambiente en términos de energía solar equivalente. Las Transformidades resultantes se compararon con el fin de poner de manifiesto que el análisis de eMergía es sensible al contexto local y los límites del sistema de referencia. Los resultados obtenidos muestran una alta dependencia en la producción de concreto sobre las fuentes de recursos externos, siendo el concreto semi-industrializado el más sustentable.

Palabras clave: análisis eMergético; contabilidad ambiental; sustentabilidad; transformidad; concreto hidráulico.

Avaliaçao da sustentabilidade de tres métodos de produçao de concreto com base no controle de qualidade

RESUMO

Este trabalho apresenta um método de avaliação ambiental na produção de concreto, baseado no consumo de recursos naturais não-renováveis. Três técnicas de produção (industrializada, semi-industrializada e manual) foram avaliadas sob o controle de qualidade eMergia. Este conceito foi utilizado para medir a quantidade do uso de recursos do meio ambiente em termos de energia solar equivalente. Os resultados foram comparados a fim de mostrar que a análise eMergia é sensível ao contexto local e aos limites do sistema de referência. Os resultados mostram uma alta dependência da produção de concreto em relação às fontes de recursos externos, sendo o concreto semi-industrializado o mais sustentável, segundo esta análise.

Palavras chave: análise eMergética; contabilidade ambiental; sustentabilidade; transformidade; concreto.

1. INTRODUCTION

In recent years, economic development has generated negative environmental consequences. There is now a pressing need to develop management tools that can help minimize environmental impacts (Berardi, 2012; Vega et al., 2013). In the construction industry, and in the industrial sector in general, integrating environmental criteria in the design and manufacturing of products can significantly reduce the environmental impact of these products throughout their life cycle-from the extraction of raw materials for their manufacturing all the way to their final disposal (Ruiz de Arbulo, et al., 2016).

Numerous studies analyze the results of different methodologies that aim to improve sustainability in construction. These methodologies include green methodology, energy models, environmental declarations of product based on Life Cycle Analysis, environmental valuations, among others (Hamza y Horne, 2007). There are numerous publications regarding applied emergy analysis some of which are presented in Table 1.

Table 1. Different applications of Emergy Analysis

Author/Year	**Application**
C. Ferreira, S. Hurtado/2010; F. Suca, A. Suca y J. Siche/2014; N. Aguilar, J. Alejandre y R. Espinoza/2015	Agroforestry Studies
E. Ortega, C. Vallim y P. Del Pozo/2014	Evaluation of Productive Systems
I. López y J. Rodríguez/2013	Analysis of environmental sustainability at the municipal level.
H. Mu, X. Feng y K. H. Chu/2012	Calculating emergy flows in production systems of complex chemical substances,
X. Wu, F. Wu, X. Tong/2015	Analysis of ecological waste recycling.
S. Bastioni, F. Morandi/2011	Energy and algebra of emergy.
T. Abel/2015; F. Morandi, D. Campbell/2015; C. Wrigth y H. Ostergard/2015; L. Zarba y M. Brown/2015	Modeling and analysis of systems.
F. Agostinho, A. Bertaglia/2015; E. Campbell/2015	Emergy analysis of energy techonologies.
B. Lacarriere, K. Deutz, N. Jamali y =. Le Corre/2015	Emergy analysis of waste recycling.
I. Li, H. Lu, D. Tilley y G. Qiu/2014; D. R. Tlley/2015; C. Vilbiss y M. Brown/2015; S. E. Tennenbaum 2015	Emergy accounting methods.

Great volumes of concrete are employed for the construction of buildings, especially for the construction of foundations, structural frames, floors, concrete slabs and prefabricated elements (Pulselli et al., 2007). Cement, the main component of concrete, is also the most consumed material in the world after water. Thus, the environmental impacts of concrete are always associated to those of cement.

World production of cement has increased constantly since the beginning of 1950, especially in developing nations. According to data from the International Cement Review (ICR), in 2012, world production of cement was 3939 million tons, which represents an increase of 8 to 9% compared to 2011. World production is estimated to duplicate in 20 years. The main producers of cement are India and China, followed closely by the United States.

In 2012 Latin America and the Caribbean produced an estimated 180 million tons of cement, with Brazil heading the list as the main producer, followed by Mexico, Colombia and Argentina. In 2012, cement consumption per capita estimated for Latin America and the Caribbean was 301 kg/capita; for Mexico, 305kg/capita. Estimates from the International Cement Review (ICR Research), place Latin American and Caribbean cement production as 4.7% of global production (2012 estimate). Per the National Institute of Statistics and Geography (INEGI. 2015) production in Mexico for the year of 2012 was of 41, 608, 413 tons.

Production of cement accounts for approximately 5% of CO_2 emissions in the world. The environmental impact caused by the emission of contaminants, particles, ashes and carbon dioxide (Kjellsen et al., 2005; Pade y Guimaraes, 2007), as well as the life cycle of both cement and concrete (Vold y Ronning, 1995; Nisbet y Van Geem, 1997), (Josa et al., 2004; Josa et al., 2007; Nazar, 2013 and the ecological footprint of concrete have been investigated in recent years (Bastianoni et al., 2007).
The Mexican Association of Independent Concrete-makers (AMCI) estimated, for the year of 2015 an eight percent growth in the sector, double the estimate of 2013, driven by federal infrastructure and housing development projects in the country. The production of concrete in Mexico stands presently at 32 million m^3 annually. Production with existing infrastructure could be 50 million m^3 annually.
Accordingly, this study refers to the intensive use of mineral non-renewable resources and fossil fuels for the extraction of inert aggregates (sand and gravel), the use of water, the production of cement for concrete mixing and, in particular, evaluates the quantity of inflows of environmental resources in the production process. Concrete mixing was evaluated as a study case, considering the standard process for the manufacturing of Composite Portland Cement 30R. In comparison with data and values calculated previously, this study shows that emergy analysis is particularly sensible to context limits and reference systems (Brown y McClanahan, 1992; Buranakarn, 1998; Björklund et al., 2001; Brown y Buranakarn, 2003; Pulselli et al., 2008). An ecological accounting method was implemented in the concrete mixing techniques included in this study with the objective of providing better comparing each technique’s sustainability.

2. METHODOLOGY

2.1 Location of the case study zone

Tuxtla Gutierrez is the capital and the largest city in the state of Chiapas, Mexico. It is the headquarters of the state government and the center of the metropolitan area. Urban growth and economic development in the city have increased with the arrival of national and foreign capital and government subsidies.
The Ease of Doing Business Index from the World Bank Group (WBG) and the International Finance Corporation (IFC), which classifies the economy of each country according to business regulations and property rights, ranks Tuxtla Gutierrez in the fifth place in Mexico. The capital of Chiapas is among the most dynamic cities in the south-southwest because of its job market, offer of goods and services, among other things.

2.2 Emergy analysis

Emergy analysis evaluates inflows and outflows of energy and materials in common units (solar emjoules, abbreviated as seJ) that allow the analyst to compare environmental and financial aspects of the system. Based on this unit, emergy can be defined as the quantity of solar energy used, directly or indirectly, to produce a particular good or service (Odum, 1971, 1983, 1988, 1996) (Brown et al., 2004). In other words, emergy is the "energy memory" that is used along a sequence of processes to obtain a good or service. Solar transformity is the solar emergy required to generate a Joule of a service or a product. Its units are the solar-emjoule/Joule (seJ/J).

The first step in the analysis is to draw a diagram of the energy-flows in the systems studied to learn what relationships exist between different components and the different resource flows. The second step consists in the construction of emergy analysis tables based on the diagrams of the previous step. In the third step, the emergy indexes that relate the flows of emergy in the economy with those in the environment are calculated. Finally, the indexes are interpreted to diagnose economic viability and the loading capacity of the systems studied.
The emergy indexes developed by Odum (1971) define sustainability referring to the quantity and quality of the energy transformed for a particular production system. Odum's analysis involves the use of the diagram three branch (see Figure 1) which facilitates calculation of the indexes.

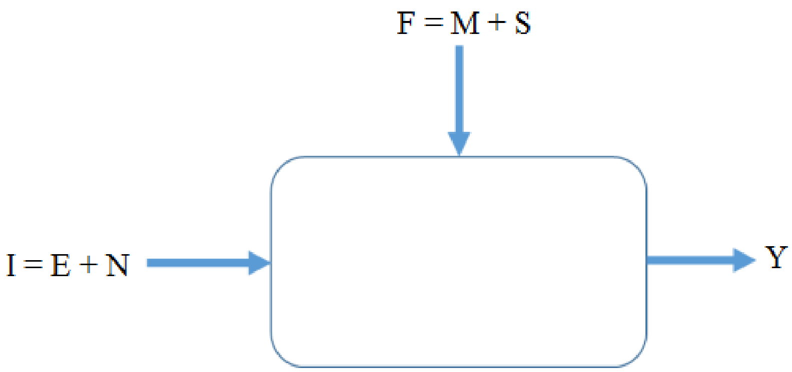

Figure 1. Three branch diagram (Odum, 1971)

Where:
I: Natural Renewable Resources (R) plus Natural Nonrenewable Resources (N)
F: Materials (M) plus Services (S)
Y: System output.
Taking the region into account, the use of nonrenewable resources by the construction industry was evaluated by means of an emergy analysis. This procedure followed a top to bottom approach beginning with an emergy analysis of the country (Mexico), then the state of Chiapas, the municipality of Tuxtla Gutierrez, and finally at the level of each technique studied. The objectives of this study are presented as follows:

- Provide a methodology that evaluates the costs and benefits of the concrete mixing.
- Compare different concrete mixing techniques using emergy indexes such as the Emergy Investment Ratio (EIR), Emergy Yield Ratio (EYR), Environmental Loading Ratio (ELR), Emergy Sustainability Index (ESI), and Transformities.

According to the information collected in the study case zone and to the present knowledge of the field of emergy analysis, three techniques were identified for the production of hydraulic concrete. In addition, the local construction industry referred to f´c=24.53 MPa concrete as the most used in the zone.
For the study of the alternatives related to concrete mixing three techniques, 1) industrialized 2) semi-industrialized and 3) manual, were selected and evaluated by means of an emergy analysis. The comparison was done using the emergy indexes obtained for each case study in the city of Tuxtla Gutierrez, Chiapas, Mexico, to determine the environmental viability of each of these options. The unit of evaluation was the cubic meter.

The following specifications were considered for each technique:

- Industrialized technique: This technique can be distinguished from the other two techniques by the use of heavy machinery, such as concrete plants, mixing trucks and front loaders, with eight workers required for the whole production process.
- Semi-industrialized technique: The main characteristic of this technique is the use of simple machinery (portable drum mixer) combined with manpower (a group of about ten workers). Helped by a gasoline powered concrete mixer, this technique is used for the mix of moderate volumes (no more than 45 m^3 by group of workers).
- Manual technique: This technique uses about fifteen workers with moderate skills, including two technicians, one of them directing the workforce and the other supervising quality. With help of simple tools and without the use of fuels, electricity or mechanical energy, this method is generally employed for small volume production (no more than 10 m^3 per group of workers).

For this study, the transformities for the inert aggregates (gravel and sand), cement and water were calculated for the city of Tuxtla Gutierrez, Chiapas, Mexico. The valuation was done taking into account an annual production of 70,000 m^3 for industrialized concrete, and a daily production for semi-industrialized and manual concrete of 45 m^3 and 10 m^3 respectively. The transformity of cement was based on the annual production of 2,190,000 tons of the local cement plant. The transformity employed for sand corresponds to the extraction area of the Santo Domingo River located in the municipality of Chiapa de Corzo, Chiapas, at a distance of 30 km from the city of Tuxtla Gutierrez, Chiapas. The crushed gravel was extracted from the quarry located in the neighborhood of Plan Chiapas in the municipality of Chiapa de Corzo where it borders with the municipality of Tuxtla Gutierrez.

The emergy indexes employed in the study were: the EIR, calculated as the relationship between contribution from the economy (F) and nature (I), which is adimensional. The EYR, is considered as the relationship between the total emergy that enters the system (Y) and the contribution to the economy (F). This index is adimensional. The higher the value of this index is, the higher the impact of the system on the global economy. ELR was calculated as the relationship between the sum of non-renewable natural resources (N) and economical resources (F) multiplied by all natural renewable resources (R). It is also adimensional. The ESI shows the contribution of the natural environment, that is to say, the energetic work that the ecosystems do to generate processes for the environmental load. This value was calculated dividing the contribution of nature (EYR) by the environmental load (ELR).

Finally, the calculated emergy values of concrete in this study were compared with those in previous emergy studies with the objective of showing how emergy analysis is sensible to local context and the limits of the reference system.

3. RESULTS AND DISCUSSION

This study allowed a comparison between the different concrete mixing techniques selected, using emergy indicators such as EIR, EYR, ELR and ESI. The emergetic attributes of each system were quantified and used as indicators of the characteristics of each technique. For the evaluation of each alternative, the concrete considered was f´c= 24.53 MPa. These proportions were obtained at the Concrete Technology Laboratory at the Engineering Faculty of the Autonomous University of Chiapas, for the study cases of manual and semi-industrialized techniques. The corresponding indicators for industrialized concrete were obtained from a concrete factory in the city of Tuxtla Gutierrez, Chiapas, Mexico.

The diagram of energy flows interacting within the concrete mixing system shows (Figure 2) renewable and nonrenewable resources as well as the energy acquired by inputs (materials, services, manpower). Based on the flow diagrams, an emergy analysis was performed for each alternative and is presented in Tables 2, 3 and 4. For each case, the emergy of concrete mixing was based on the following supplies: raw materials, transportation, equipment and machinery, fuels, manpower, maintenance and insurance.

The total emergy consumed by each mixing technique was of 5.9 E15 seJ, 5.87E15 seJ and 8.32E15 seJ, with manual means, semi-industrialized and industrialized, respectively as can be seen on Tables 2, 3 and 4. In the mixing of industrialized concrete, 98.14% of the total emergy is materialized in the sedimentary natural cycles of construction materials. Machinery accounts for (0.084%), fuel for (1.00%) and manpower for (0.24%). With semi-industrialized concrete, 99.44% is materialized during the sedimentary natural cycles of construction materials. Equipment and tools account for (0.045%), fuels (0.022%) and manpower (0.076%). Finally, concrete mixed by manual means shows that 98.02% of the total emergy is materialized in the natural sedimentary cycles of the construction materials. Equipment and machinery account for (1.47%), and manpower for (0.50%).

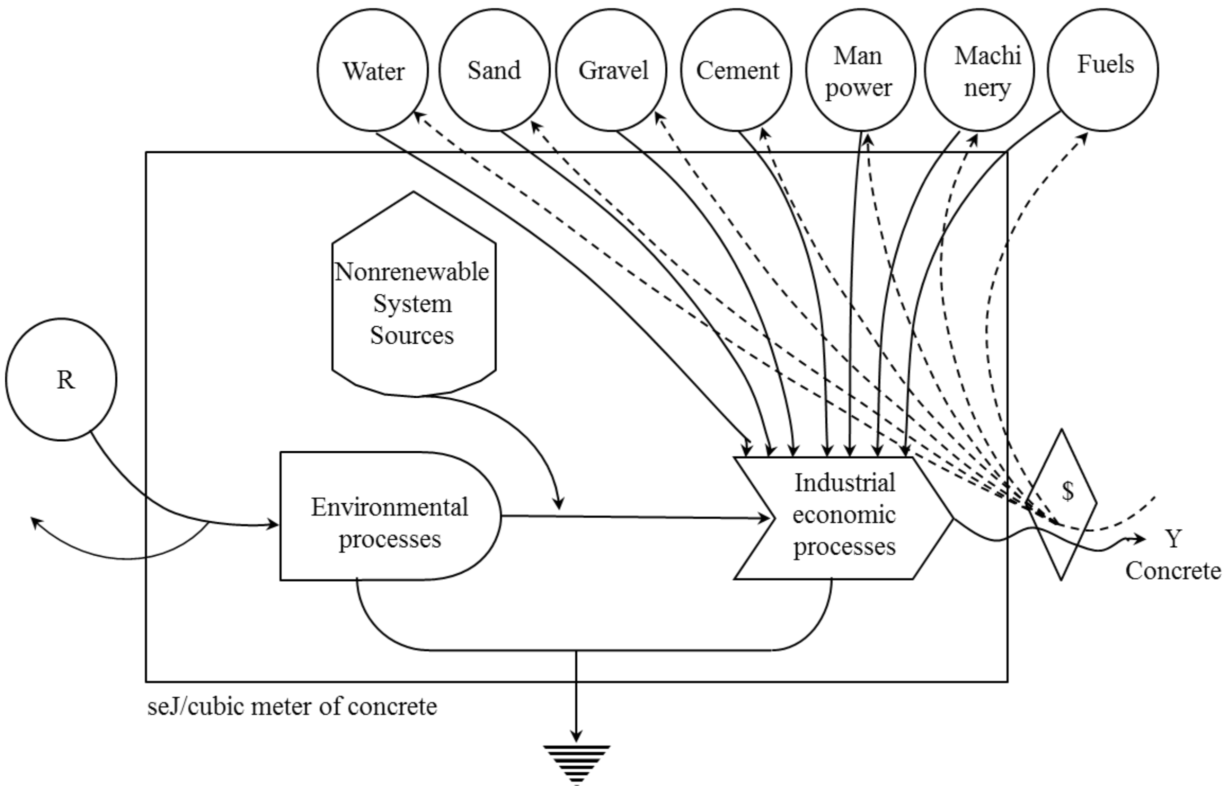

Figure 2. Simplified diagram of energy flows in the production of concrete.

Table 2. Emergy analysis of the production of industrialized concrete

No.	Description	Units (unid/m³)		Transformity (seJ/unid)	Emergy (seJ/m³)	References
	MATERIALS					
1	Cement	4.00E+05	g	3.61E+09	1.44E+15	*
2	River Sand	1.07E+06	g	3.29E+09	3.53E+15	*
3	Crushed gravel T.M.A. 3/4"	1.42E+06	g	2.24E+09	3.19E+15	*
4	Water	2.51E+05	g	3.27E+06	8.19E+11	*
5	Diesel	1.08E+08	J	6.60E+04	7.13E+12	Doherty et al, 1994
6	Lubricants	3.21E+05	J	6.60E+04	2.12E+10	Doherty et al, 1994
7	Electric power	2.76E+08	J	2.77E+05	7.63E+13	Odum, 1996
	PLANT AND MACHINERY					
8	Concrete doser	1.86E+02	g	6.70E+09	1.25E+12	Doherty et al, 1994
9	Cement Silo	8.06E+01	g	6.70E+09	5.40E+11	Doherty et al, 1994
10	Aggregate hopper	5.58E+01	g	6.70E+09	3.74E+11	Doherty et al, 1994
11	Conveyor belt	3.10E+01	g	6.70E+09	2.08E+11	Doherty et al, 1994
12	Cement weighing machine	2.23E+01	g	6.70E+09	1.50E+11	Doherty et al, 1994
13	Aggregate weighing machine	3.47E+01	g	6.70E+09	2.33E+11	Doherty et al, 1994
14	Water doser	2.48E+01	g	6.70E+09	1.66E+11	Doherty et al, 1994
15	Front loader	4.30E+02	g	6.70E+09	2.88E+12	Doherty et al, 1994
16	Mixing truck	1.14E+02	g	6.70E+09	7.64E+11	Doherty et al, 1994
17	Dump truck	5.95E+01	g	6.70E+09	3.99E+11	Doherty et al, 1994
	SERVICES					
18	Manpower	4.19E+06	J	4.77E+06	2.00E+13	Guillén, 1998
19	Maintenance and insurance	9.46E-01	$	4.59E+13	4.34E+13	*

*Transformity calculated for this study.

Y = 8.32E+15

Table 3. Emergy analysis of the production of semi-industrialized concrete

No.	Description	Units (unid/m³)		Transformity (seJ/unid)	Emergy (seJ/m³)	References
	MATERIALS					
1	Cement	4.02E+05	g	3.61E+09	1.45E+15	*
2	Sand	7.25E+05	g	3.29E+09	2.38E+15	*
3	Crushed Gravel T.M.A. 3/4"	9.04E+05	g	2.24E+09	2.02E+15	*
4	Water	2.33E+05	g	3.27E+06	7.60E+11	*
5	Gasoline	1.92E+07	J	6.60E+04	1.27E+12	Doherty et al, 1994
6	Lubricants	3.33E+04	J	6.60E+04	2.20E+09	Doherty et al, 1994
	EQUIPMENT AND TOOLS					
7	Wooden shovel handle	3.82E+03	g	6.79E+08	2.59E+12	Odum, 1996
8	Spoon shovel with metallic handle	8.77E-01	g	3.16E+09	2.77E+09	Bargigli et al, 2003
9	Plastic container with 19 liters capacity.	1.73E+00	g	8.57E+04	1.48E+05	Brown et al, 2003
10	Portable mixer with 2 sacks capacity	7.42E+00	g	6.70E+09	4.97E+10	Doherty et al, 1994
	SERVICES					
11	Manpower	9.30E+05	J	4.77E+06	4.44E+12	Guillén, 1998
12	Maintenance and insurance	6.36E-05	$	4.59E+13	2.92E+09	*

*Transformity calculated for this study

Y = 5.87E+15

Table 4. Emergy analysis of the production of concrete by manual means

No.	Description	Units (unid/m³)	Transformity (seJ/unid)	Emergy (seJ/m³)	References
	MATERIALS				
1	Cement	4.02E+05 g	3.61E+09	1.45E+15	*
2	Sand	7.25E+05 g	3.29E+09	2.38E+15	*
3	Crushed gravel T.M.A 3/4"	9.04E+05 g	2.24E+09	2.02E+15	*
4	Water	2.33E+05 g	3.27E+06	7.60E+11	*
	EQUIPMENT AND TOOLS				
5	Wooden shovel handle	1.29E+05 g	6.79E+08	8.76E+13	Odum, 1996
6	Spoon shovel with metallic handle	2.96E+01 g	3.16E+09	9.35E+10	Bargigli et al, 2003

7	Plastic container with 19 liter capacity.	1.17E+01 g	8.57E+04	1.00E+06	Brown et al, 2003
	SERVICES				
8	Manpower	6.28E+06J	4.77E+06	3.00E+13	Guillén, 1998

*Transformity calculated for this study **Y = 5.98E+15**

The EIR value of industrialized concrete is higher when compared with the same value for semi-industrialized concrete and concrete produced by manual means. The value stands at 10,161 for industrialized concrete, at 7,724 for semi-industrialized concrete and at 7,867 for concrete mixed by manual means. This value suggests a weak competitive capacity due to the instability of external sources of materials. Figure 3 shows a comparison of the EIR for the three different techniques.

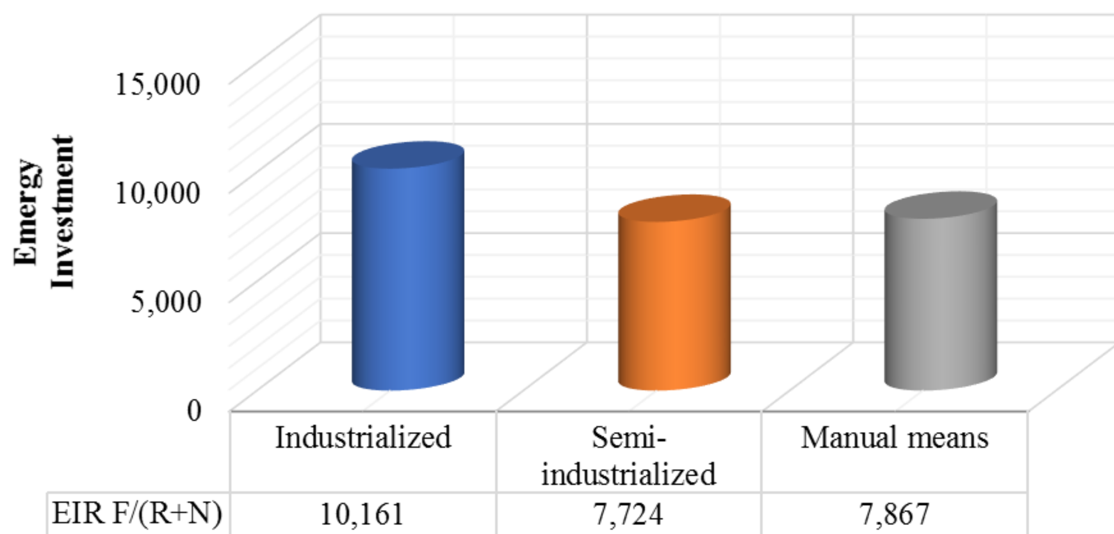

Figure 3. Emergy Investment Ratio of concrete mixing.

The EYR shows the contribution of nature to the productive system, or in other words, quantifies the work of the ecosystem to obtain products. The techniques analyzed in the study presented a value lower than 1.00, which indicates that the emergy liberated by these systems is equal to that invested in the form of economic resources. In other words, it implies that they are highly dependent on imported supplies and services. An EYR value higher than 1 indicates that the system analyzed generates more new resources (of emergy) than those available as inputs, otherwise, the system is a transforming consumer of resources. In Figure 4 the production of emergy in concrete has been graphically illustrated.

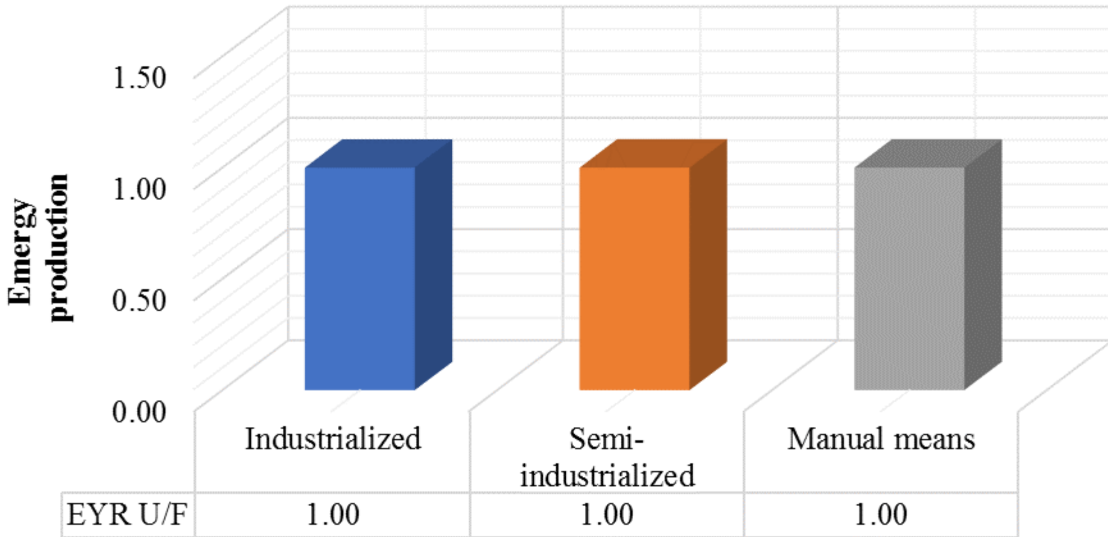

Figure 4. Emergy production in the manufacturing of concrete.

The analysis shows that industrialized concrete has a higher ELR, with a value of 10,161 compared to the values obtained for semi-industrialized and concrete produced by manual means (7,724 and 7,867, respectively). These values are relevant to demonstrate the degree to which concrete mixing is harmful towards the environment. It is important to note that concrete produced by manual means presents a higher environmental load than semi-industrialized concrete due to low productivity. This is evident when you consider the 10 m^3 produced in 8 hours of work by manual means, compared with 45 m^3 for semi-industrialized concrete. The ELR reflects the energy incorporated to the system by the manpower employed in both techniques. The Environmental Load of both techniques is compared in Figure 5.

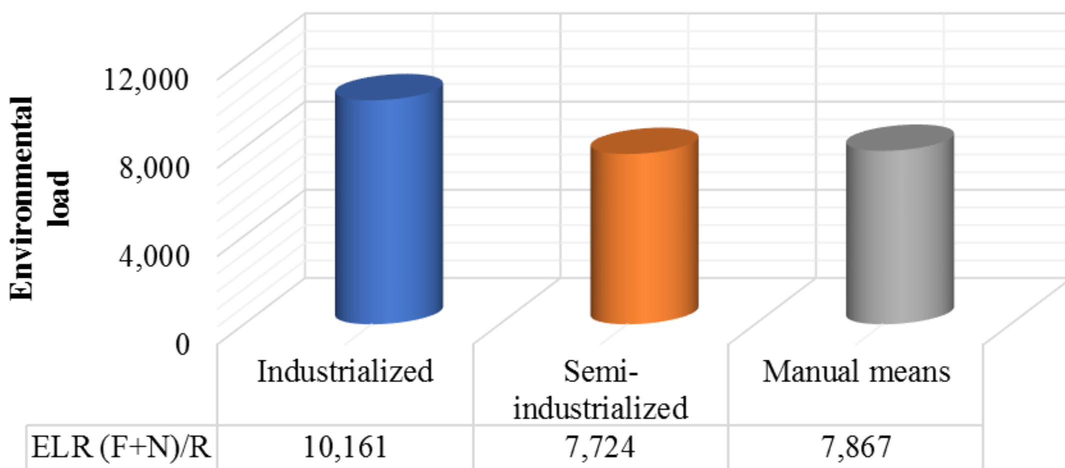

Figure 5. Environmental load exercised on the environment by concrete manufacturing.

The ESI indicates the contribution of the natural environment, meaning that the energy work that is done by ecosystems to generate processes that act upon the environmental load. According to Brown and Ulgiati (2004), ESI values inferior to 1 indicate systems that consume resources and are associated with highly developed, consumption-oriented economies. The values reported in this study indicate that semi-industrialized concrete (0.000129) has a higher ESI value than that of concrete produced by manual means (0.000127) and industrialized concrete (0.0000984). This means that semi-industrialized concrete is the one that impacts environmental equilibrium to a lesser degree, and is therefore more sustainable for the environment than industrialized concrete and concrete produced by manual means. In relation to this index it is important to note that the main difference between semi-industrialized (4.44E+12) and manual production (3.00+13) is the manpower employed. Figure 6 shows a graphic comparison of the ESI results obtained for each alternative.

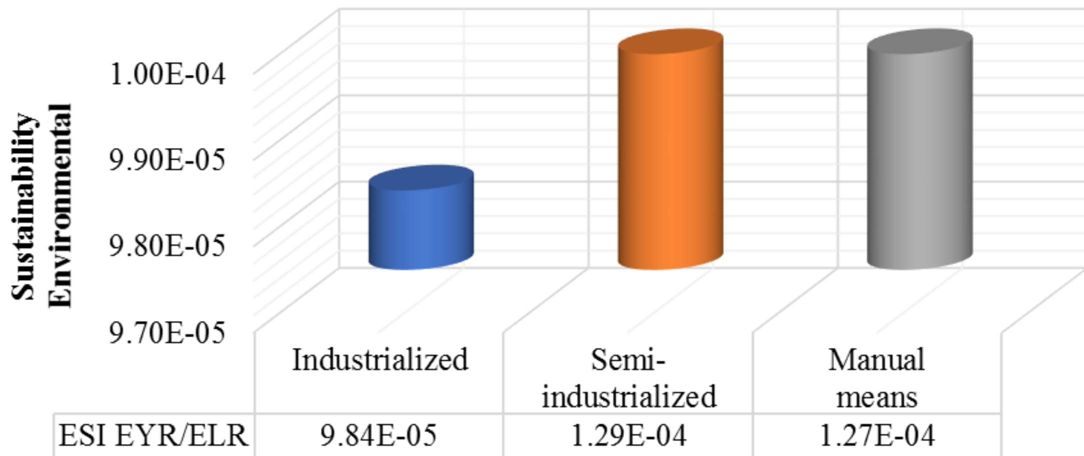

Figure 6. Sustainability in the production of concrete

With relation to other studies conducted in reference to the mixing of industrialized concrete, the following significant differences were found:
In Bjorklund et al (2001): the authors used solar transformity values in a national context. For example, the transformity of electricity is very specific, because it was evaluated according to production processes in Sweden, which includes nuclear energy (33%- here represented with the transformity of median world electric production), and hydroelectric energy (66%). In the present study, the emergy evaluation of cement production is not considered in the whole production process, and this represents an important approximation. In regards to the main supplies used in concrete mixing, the present study only considers the use of limestone, electricity and petroleum, while other inputs such as transport, packaging and services like manpower machinery and fuel were not evaluated. Electricity and manpower were evaluated by means of an emergy/money relation.
In Buranakam (1998), specific emergy was evaluated for the United States. The analysis included an evaluation of highways, vehicles and infrastructure used, taking into account the whole national transportation system. The author calculated the total length of national highways and their production process (materials, energy, manpower and other services), taking into account the annual cost of construction. This value (in seJ/km) was divided by the percentage of buses in relation to the total weigh of vehicles (cars, buses, trucks, and others). The same was done with railroads and maritime services (boats). In general, the useful life of highways, vehicles and all relevant infrastructures was not taken into account. Manpower and other services were evaluated using an emergy/money relation.
In the work of Brown and Buranakam (2003), the emergy evaluation was done based in Buranakam 1998, and it took into account the different stages in the use of materials, demolition and reuse. Accordingly, their analysis had to do with process inputs that had to do with specific

procedures. The present work had the objective of determining the transformity of concrete, from its origin, to its use in the construction of buildings- excluding the disposal stage.
Similarly, Brown and McClanahan (1992) evaluated the specific emergy of Thailand with reference to the energy sources available at the local level. This analysis was carried out in 1992, using data from 1983. The emergy analysis was very simplified in comparison to the one in the present study due to the fact that the authors considered some problems such as material flows, petroleum and electricity as energy flows and other goods and services, this last one evaluated in terms of money flow (by means of an emergy/money relation).
On the other hand, Pulselli et al. (2008) centers his attention on the production process of the Italian cement and concrete industry. Most the values of the transformities in the study were evaluated in the U.S.A. The authors carried out an evaluation of a specific process taking into account a specific amount of 23 tons of concrete and its transportation to the construction site. In the selection of this system they consider some insignificant factors such as the evaluation of the total national transportation infrastructure (highways and other services). Figure 7 compares the transformities of concrete obtained by the authors previously cited and compares them with the results obtained in this work.

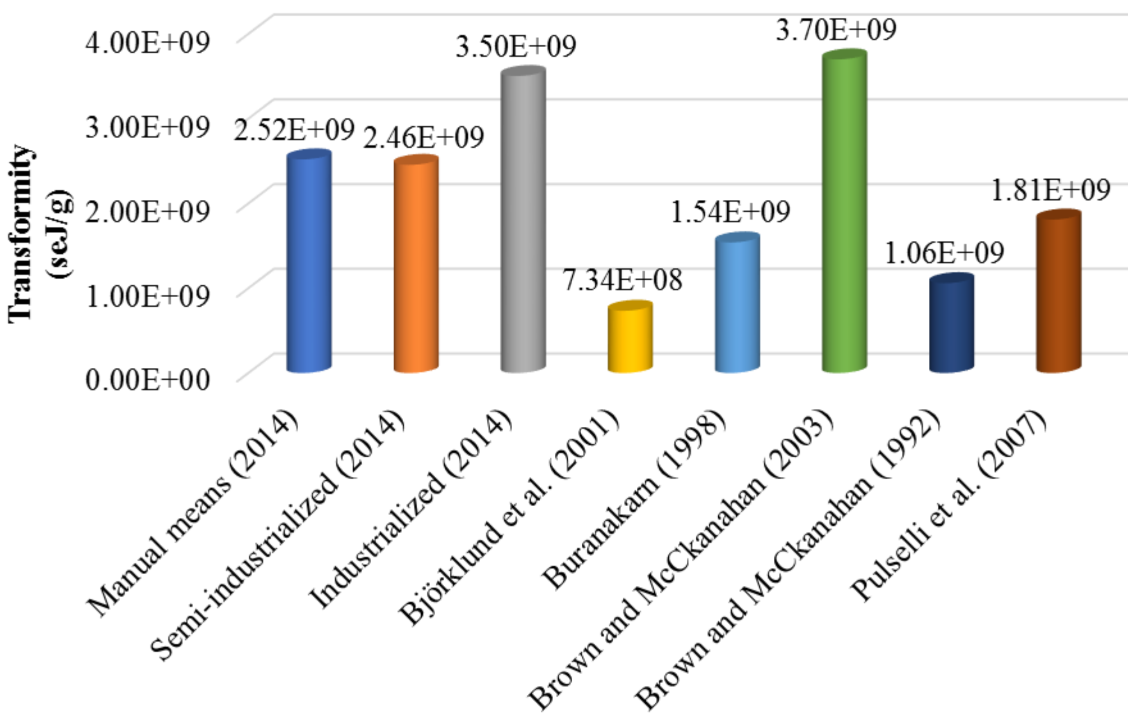

Figure 7. Comparison of the transformities of concrete.

Emergy analysis of concrete mixing based on quality control considers several steps in the process, from the production of materials to the concrete mixing process. The results of this study show that the impact of quarry materials on the total emergy is higher than the impact of production processes. Quarry materials, product of natural sedimentary cycles and counted in seJ, present a very high transformity. The present analysis shows that excessive use of non-renewable resources is critical for the construction industry because it considers not only human work done in quarries (process which is included in the economical analysis), but also the work done by nature (sedimentary cycle). The important part played by mineral resources in the production of cement highlights how unsustainable the construction industry is.

Transformity is presented as a measurement of the use of environmental resources in the form of materials used during construction. It provides a classification of construction materials based on an energy hierarchy. Emergy analysis combines quality (Transformity) with quantity (energy or mass). The environmental impact of the construction process will depend on the materials used (in terms of environmental cost due to energy use and mass) and on the process itself (quantity of materials needed to construct structural elements). Transformity shows that the main difference between semi-industrialized (4.44E+12) and manual manufacturing techniques (3.00+13) is the manpower employed. Consequently, this index can be taken as a measurement of the use of natural resources in the form of construction materials and can also help when creating a classification of construction materials according to an energy hierarchy.

According to Buron Maestro (2012), sustainability does not have an absolute value. Its purpose as a concept is to help compare and select the most appropriate options so that the present development of society can continue without compromising the development of the future. There are no sustainable solutions; there are only solutions that are more sustainable than others.

4. CONCLUSIONS

The emergy method allowed for an analysis that considers the relationships between natural systems and anthropic activities and helps us look for strategies that are ever more sustainable.

The EIR was evaluated as an indicator of sustainability by measuring dependency on local or external sources. It was demonstrated that concrete mixing processes depend to a high degree in supplies obtained from outside the system (emergy input from imports).

The ESI was employed as a measure of the contribution of the system hierarchically higher to the production of the system by load unit of itself. The results show that semi-industrialized concrete is more sustainable than concrete manufactured by manual means or by industrial processes.

Emergy analysis provides a way to measure the sustainability of concrete mixing techniques based on quality control in terms of energy investment. Many units of low quality energy are used to provide high quality energy (high Transformity). Energy is materialized by means of a chain of transformative processes and its memory is conserved by the production structure.

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Rebuilding Nepal for next earthquake

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ABSTRACT
The paper is prepared to draw attention of local and international community including the government and donors to gear up for policy reform and create an environment for investing in proactive earthquake safety initiatives before the next earthquake strikes. The paper focuses on the outcome of the author's continuous interaction with local community since 1985 on the need for extended earthquake safety initiatives through stakeholders' easy access to technical assistance and financial resources. The most neglected aspect in the earthquake initiatives of Nepal is the lack of state ownership and lack of dedicated responsible institutions resulting in a massive toll of life and property. It is time to use the opportunity created by the April 2015 earthquake.
Keywords: policy reform; proactive initiatives; conservation; strengthening.

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Reconstruyendo Nepal para el siguiente terremoto

RESUMEN

El documento está preparado para llamar la atención de la comunidad local e internacional, incluyendo el gobierno y los donantes, para prepararse para la reforma de políticas y crear un ambiente para invertir en iniciativas proactivas de seguridad ante terremotos antes del próximo terremoto. El documento se centra en los resultados de la interacción continua del autor con la comunidad local desde 1985 sobre la necesidad de ampliar las iniciativas de seguridad de terremotos a través del fácil acceso de los interesados a asistencia técnica y recursos financieros. El aspecto más desatendido en las iniciativas con respecto a terremotos en Nepal es la falta de propiedad estatal y de instituciones responsables, lo que da lugar a un gran número de pérdida de vidas y bienes. Ya es hora de utilizar la oportunidad creada por el terremoto de abril de 2015.

Palabras clave: reforma de políticas; iniciativas proactivas; conservación; fortalecimiento.

Reconstruindo o Nepal para o próximo terremoto

RESUMO

Este artigo é elaborado para chamar a atenção da comunidade local e internacional, incluindo o governo e os doadores para se preparar para a reforma política e criar um ambiente para investir em iniciativas pró-ativas de segurança de terremoto antes do próximo terremoto se iniciar. O artigo enfoca os resultados da interação contínua do autor com a comunidade local desde 1985 sobre a necessidade de iniciativas de segurança ampliada de terremoto através do fácil acesso das partes interessadas à assistência técnica e recursos financeiros. O aspecto mais negligenciado nas iniciativas para prevenção contra terremotos do Nepal é a falta de propriedade estatal e a falta de uma instituição responsável neste assunto, resultando numa perda maciça de vidas e prejuízos às propriedades. É tempo de aproveitar a oportunidade criada pelo terremoto de abril de 2015.

Palavras-chave: reforma política; iniciativas proativas; conservação; reforço.

1. INTRODUCTION

The Nepal earthquake of April 25, 2015 and two major aftershocks of April 26 (magnitude 6.1) and of May 12, 2015(magnitude 6.8) and 425 smaller aftershocks (magnitude over 4) has left Nepal devastated making it difficult to return to normal life. Perhaps, the meaning of devastation is fully revealed in the experience of this quake, which has disrupted urban and rural physical settings and besides destabilizing mindsets as shown by reports from all over the world in vivid color.

Many aid workers were frustrated due to inability to visualize aid delivery to needy communities in the hinterlands and supply to more accessible urban areas including the Airport. Many supplies below national or international standards were dumped openly in the Airport and could not get into the country, a real pathetic scenario. The first few days saw many people flee of the country in panic in selfish disregard for the local partners with whom they had shared so much. Many countries rescued their own people leaving others to despair. Scenes of some running away while others rushing in led one to ponder over the wisdom of the rationale of action itself.

The rescue of devastated people from under the rubble in the aftermath of the Earthquake was a spontaneous efforts of local people and authorities working without any proper instructions – the Red Cross and local volunteers were much appreciated for help in rescue of several lives from the rubble. It was not surprising that those at the top floors escaped the death traps. The prompt relief by the international and the local communities were what brought the quake ravaged populace to a safe mode of refuge in the temporary shelters like tents, tarpaulins and tunnels of corrugated sheets.

This helped society in turn to achieve some resilience to earthquake by ensuring that post-earthquake epidemics like cholera, typhoid, swine flu, dysentery and diarrheria do not occur. Spontaneous volunteerism and active SMS network across the country warned about the range of precautions needed to be taken in a state-of–the-art show of our performance.
Cities in Nepal after the April quake look normal and did not at all resemble quake stricken cities. Vital infrastructure like water supply, electricity, telecom, roads, bridges, and airports remained unaffected and services were not disrupted. That was instrumental in effective delivery of the international and domestic relief works across the affected 14 districts. However, Kathmandu-Kodari Road, a vital link with China across Mahabharat and Himalayan range, was severely damaged and remained unserviceable. The surprise that this was not reported by the government and media is a proof of miss governance.
The damages though accounted as significant did not match that forecasted by previous studies (UNDP, 1992). The estimated and actual casualties and damages are presented in Table 1 below.

Table 1. Damages and toll

Description	Expected toll	Actual toll	As % of national figure
Human toll	100,000	8,969	0.03%
Injuries	300,000	22,321	
Collapsed buildings in Nepal	546,000	893,539	8.33%
Fully /partially damaged private houses		887,074	4.46%
Fully /partially damaged health facility		963	3.33%
Government offices		6,465	
Schools		6,308	
Industries		133	
Collapsed/damaged cultural heritage		745	
Endangered cultural heritage		1500?	
Hydropower damaged		18	
Bridges	> 50%	1	0.07%
Roads	> 10%	Few places	Very small
Water supply	> 95%	Few days	Very small
Telephone	> 60%	None	None
Source: Kathmandu Valley Earthquake Risk Mapping Project, UNDP 1992;			
http://drrportal.gov.np			

Apart from damaged buildings making over 4.5 million people homeless, numerous landslides and rock falls were triggered in the mountain areas, temporarily blocking roads.
The 1934 Bihar-Nepal Earthquake produced strong shaking in the Kathmandu Valley, destroying 20 percent and damaging 40 percent of the Valley's building stock. In Kathmandu, itself, one quarter of all homes were destroyed along with several historic sites (USGS).
The current Kathmandu cityscape is hardly indicative of one stricken by an earthquake. This is the result of 30 years' hard work of many people preparing in advance against the hazards of earthquakes. Damages and casualties were minimal because of this hard work without precedence. The airport was running 24/7; all bridges were intact, emergency supplies undisturbed; high rise buildings still standing tall in spite of non-structural cracks all over, and thousands of houses, commercial and institutional buildings standing intact except those that compromised on quality; devastation all around but people still smiling.

2. PROBLEMS AND ISSUES

The huge toll of life and property in the April 25, 2015 and numerous aftershocks could have been reduced considerably if capacity building of the local community, government and non-government agencies had been undertaken in time and a dedicated agency given charge. It was well known to all that a large earthquake was overdue and the only way to face such earthquakes is to make adequate preparations. Glaringly visible tasks such as need for updating building codes and urban development bylaws, removing the weaknesses and mischief in them, putting sincere efforts in implementation of the bylaws and codes, checking the strengths of buildings and determining the design earthquake, the need for peer review of design, quality and construction, verification, certification and the like were knowingly or unknowingly neglected and not implemented.
In spite of several voices called for attention to need for declaring policy on building Earthquake Safer cities and protecting important premises like historic cultural monuments, schools, hospitals, industries, communication and tourism infrastructure, the country has no pronounced program to the effect needed. Priorities related to conservation of heritage and cultural values versus modern engineering technology needs to be established. The technology for safeguarding millions of existing structures needs to be identified. The encouragement and motivation factors for investment in earthquake safer cities are still missing.
The need for training of municipal and practicing engineers in the design and construction of small buildings was initially addressed through young engineers training for earthquake resistant design with the support of UNDP (UNDP/Earthquake Safety Initiatives, 2008) but lately discontinued due to lack of support and initiatives.
This deplorable situation cannot continue any more. There is a strong felt need to find ways to create earthquake resilient communities through credible institutions, coordinated programs, environment for effective delivery mechanism, checking and verification of the actual deeds, and assuring the plans and programs are effectively implemented.

3. THE OBJECTIVES

The objectives of the paper are:

- To draw attention of the local and international communities to make significant investment in capacity building of the country as a whole to face challenges of potentially large earthquakes in the future,
- To strengthen 5.5 million units, already weakened by the current earthquakes and aftershocks, and comprising mostly Brick/Stone construction in mud mortar,
- To draw attention to the need to set a target for the next earthquake: the human toll below 1,000!
- To draw attention of the community and the government on the need for recovery and conservation of lost cultural heritage and ancient heritage settlements as priority, recovery of vast urban and rural settlements, and help to conserve and regenerate local economy to sustain the post-earthquake recovery needs,
- To provide training to structural engineers, architects and urban planners for post-earthquake recovery, seismic resistant planning and construction, and artisans training for quality construction,
- To encourage documentation of all premises for assuring earthquake safety,
- To help develop recovery guidelines,
- To help update building bylaws and building codes based on the lessons learnt from the recent earthquakes and international experience, and

- To draw attention to the need to establish an apex agency for earthquake affairs to develop ownership and responsibility.

4. THE GRAND REHEARSAL OF FUTURE EARTHQUAKES

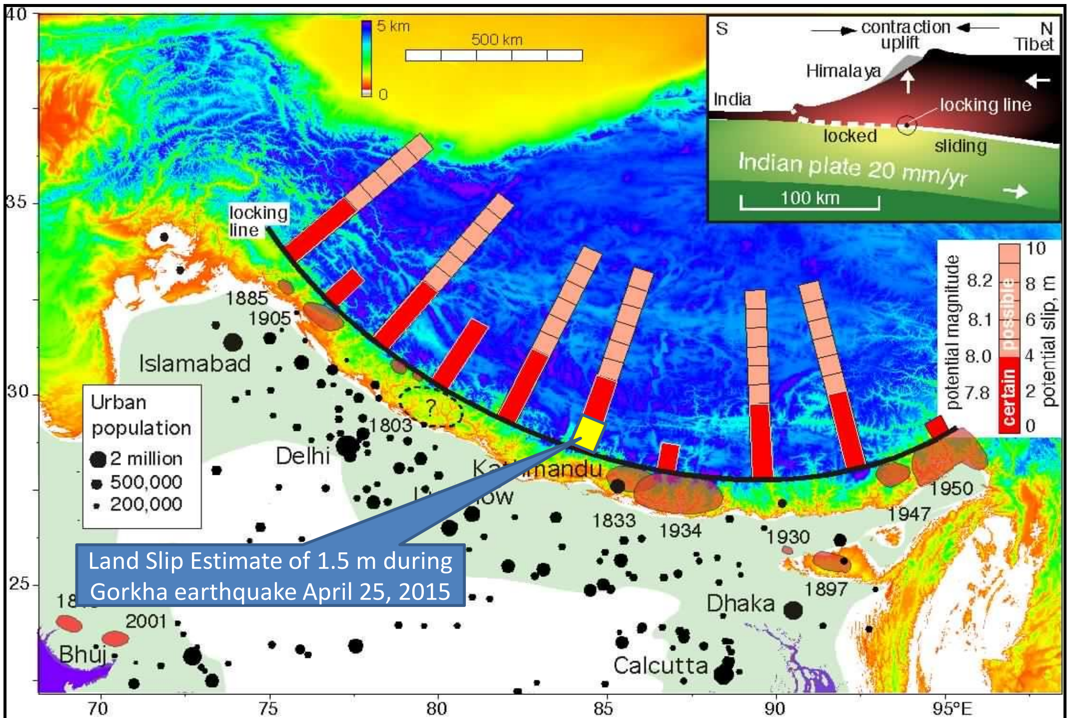
The potential for earthquakes in Nepal was already realized immediately after the 1988 Earthquake of Dharan and Rajbiraj which killed 722 people in Nepal and India, injured 12,000 and 450,000 left homeless. The best part of this quake was the triggering of awareness within the Government in Nepal and the donor communities leading to the establishment of the Kathmandu Valley Earthquake Risk Management Project, 1997.

The USGS quick report on the April 25, 2015 Gorkha Earthquake made reference to very large Nepal earthquakes, with a moment magnitude of 7.5 or more, observed in the historic periods in 1100, 1255, 1505, 1555, 1724, 1803, 1833, 1897, 1947, 1950, 1964, 1988. Three earthquakes comparable to the Gorkha Earthquake occurred in the Kathmandu Valley in the 19th Century: in 1810, 1833, and 1866. Seismic record of the region, extending back to 1100, suggests that earthquakes of this size occurred approximately every 75 years, indicating that a devastating earthquake is inevitable in the long term.

The strong motion network of Nepal is quite limited. Nevertheless, Kanti Path (Kathmandu) recorded the maximum ground acceleration of 0.164 g. The USGS preliminary estimation of the maximum ground acceleration (PGA) in the epicenter area was about 0.35g and 0.1 - 0.15 g for Kathmandu. In Western Nepal, PGA range was between 0.5 g and 0.6 g, whereas in Eastern Nepal that ranged between 0.3 g and 0.6 g. The PGA estimate was based on the empirical relations developed by Aydan (Aydan and Ohta, 2011; Aydan 2007, 2012).

Mr. Jean Ampuero, California Institute of Technology, in his paper "Salient Features of the 2015 Gorkha, Nepal Earthquake in Relation to Earthquake Cycle and Dynamic Rupture Models" indicates that the high-frequency (HF) ground motions produced in Kathmandu by the Gorkha Earthquake were weaker than expected for such a magnitude. The static slip reached close to Kathmandu but had a long rise time. An important observation (Katsuichiro Goda, Department of Civil Engineering, University of Bristol, Bristol, UK et al) is that the ground motion shaking in Kathmandu during the 2015 main shock was less than the PGA estimates (with 10% probability of exceedance in 50 years i.e., a return period of 475 years). This may indicate that ground motion intensity experienced in Kathmandu was not so intense, compared to those predicted from probabilistic seismic hazard studies for Nepal. Therefore, a caution is necessary in relation to future earthquakes in Nepal because the 2015 earthquake is not necessarily the worst-case scenario and more intense Earthquakes may be in the making.

The surface deformation measurements including Interferometric Synthetic Aperture Radar (InSAR) data acquired by the ALOS-2 mission of the Japanese Aerospace Exploration Agency (JAXA) and Global Positioning System (GPS) data were inverted for the fault geometry and seismic slip distribution of the 2015 Mw 7.8 Gorkha Earthquake in Nepal. The rupture of the 2015 Gorkha earthquake was dominated by thrust motion that was primarily concentrated in a 150-km long zone 50 to 100 km northward from the surface trace of the Main Frontal Thrust (MFT), with maximum slip of ~ 5.8 m at a depth of ~8 km, and 1.5 m at surface in Kathmandu Valley. In 1988, Roger Bilham estimated this slip would be of magnitude of at least 10 m (Figure 1). Thus, based on the observed values of maximum land slip and the maximum Probable Ground Acceleraation (PGA), the April Earthquake could be termed as a grand rehearsal for bigger future earthquakes in Nepal.



Source: Roger Bilhamand Peter Molnar of the University of Colorado and Vinod K Gaur of the Indian Institute of Astrophysics, 1997

Figure 1. Earthquake Gap in Himalayan Arc

5. VULNERABILITY ASSESSMENT AND CERTIFICATION NEEDS

Most of the existing buildings stock in rural and urban areas comprises of Non-engineered traditional construction of Brick/stone in mud mortar, with some recent buildings in cement and RCC structure. In the aftermath of the April Earthquake, it is assumed that over 80% of the damaged buildings fall in the first category of brick and mud construction, and remaining buildings in second category. But there is no post-earthquake detailed vulnerability assessment report of damaged and existing building stock available at this time. However, it is absolutely necessary to determine whether the existing building stock can withstand the next Most Considered Earthquake or Design Earthquake. This question demands a detailed vulnerability assessment of the building stock covering four issues: 1) lack of documentation of the building stock, 2) Updating of building code with consideration of recommended Design Earthquake Model. Many of these buildings are not designed to sustain that kind of load; 3) construction quality and change in occupancy, and 4) maintenance (Samir Chidiac, McMaster University, May 21, 2008).

No matter how good the design is, the building is not the one it should be if it is not built or used as specified. This is what we have seen happening quite often. The buildings designed and constructed have neither the quality monitoring certificates, nor operational monitoring like design load maintenance, occupancy and maintenance certification. Even a well-built building age, which means its properties change, and we have a problem if we do not address these issues. One of the most important actions carried out in Nepal immediately after the earthquake was the rapid visual vulnerability assessment of buildings. But the action faced controversy because of lack of adequate preparation and legal provisions. The tools used were informally borrowed from ATC 40 without proper legal backup and training. Most controversial action was the issue of Stickers (Green, Yellow and Red) categorizing the buildings into Safe, Caution, Unsafe (Figure 2). The actions created confusion in the community about its rationale and relevance. Surely, that was the result of lack of preparedness for such rapid action.

Strikers: Safe and Unsafe Category of Buildings
Figure 2. Rapid Vulnerability Assessment

The stickers were a good example of lack of adequate preparation. They were issued in very unprofessional manner and illegally since there were no such laws or guidelines that provide authority to do so. The Rapid Vulnerability Assessment forms were borrowed from elsewhere without authorization, proper guidelines and did not match with typology of the buildings in the country.

6. THE CHALLENGES

6.1 Recovery of damaged buildings.

Table 1 above indicates the extent of damages to the building stock that include various category of buildings such as 1) Low rise concrete buildings, 2) Residence in brick masonry in cement mortar, 3) Residence in brick in mud mortar, 4) Residences in traditional heritage Buildings in brick and mud mortar, and 5) Rural construction in stone in mud mortar, and 6) Rural construction in bamboo and thatch roof.
The distribution of the category of these buildings is not known. There are two major challenges: 1) Demolition of collapsed buildings and disposal or reuse of debris, and 2) Rehabilitation of partially damaged buildings and buildings with minor damages. The general psyche is that buildings with cracks (whatever may be the extent and cause) are no more useful for habitation and many started demolition without any thoughts to potential for restoration or rehabilitation. That has created strain on building stock deficiency creating huge price rise on rental. But the rational for recovery is on rise.
Quick recovery of damaged buildings immediately after the earthquake was a very important aspect that would reduce the strain on the building stock. But in the absence of recovery guidelines, access to resources like technology of recovery and financing, the people gradually forgot the earthquake shock and started recovery in their one way, mostly guided by the approach to quick repair and to demonstrate that the buildings were not affected by the earthquake. They could no more wait for proper process but made efforts for quick financial recovery through early use of the premises, neglecting safety issues. The buildings demolished during the relief works period was never recorded and analyzed to find the root cause of the damages and actual effect of the earthquake.

6.2 Conserve and earn.

Most challenge is faced by the traditional residential buildings and heritage monuments with vernacular aesthetics that represented the identity of the country and carried the value of history and culture of over 2,500 years. Recovery of these buildings in the original form and shape would be a strain on resources unless specific measures are taken to recover the lost heritage and generate economic return. The traditional residences without modern infrastructure and vehicular access could be very redundant. There are several approaches being forwarded under the principles of "Integrated Settlement Development" which will be developed following massive dismantling of damaged buildings to produce an outlook (Pilachhen Integrated Reconstruction in Lalitpur and Khokana, 2016, Figure 3).

A: Pillachhen redevelopment
(Source: Maya Foundation)

B: Khokana reconstruction
(Source: Kantipur Daily)

Figure 3. Some cases of proposed recovery of traditional heritage settlements in Lalitpur.

This will be totally a new construction and will carry none of the cultural or historical values represented in the settlements. The modern trends towards quick recovery will change the landscape and will lead to the extinction of ancient values and a total loss of the whole heritage assets. The broad objectives of these reconstruction as stated are: 1) To provide safe living and healthy environment by repairing and reconstructing houses of the local residents, 2) To protect the traditional architecture, 3) To develop infrastructures and improve vehicular accessibility, 4) To promote local business, 5) To increase income of local residents by promoting the tourism - oriented business, and 6) To conduct programs in social buildings and open space for encouraging social interactions.

Though the reconstruction program has prevision for protection of traditional architecture in its objectives, it has ignored the conservation of heritage and historic values of 2,500 years. Possibly, we are wandering in the forest of post-earthquake slogans and terminologies like Building Back Better, rebuild, recovery, retrofit, renovation, rehabilitation, protection, conservation and reconstruction. Until we are clear about our needs, we are sure to be swept away by the flood of funds being poured in the reconstruction. Immediate resource mobilization and the money power it represents is much stronger than represented by professionals struggling with lack of resources and time. However, sharing information on best practices may be still relevant and useful if only to give some lessons and directions for future. Some of the examples of regeneration based on recovery of cultural heritage settlements promoted under the principle of "Conserve and Earn" have successfully carried the message for paying attention to heritage conservation. These schemes are very popular and are better known as "Home Stay" tourist accommodation. Some of the better examples are: Shrestha House and Swotha Café (Figure 4).

Figure 4. Shrestha house and Swotha Café converted to “Conserve and Earn” projects

The innovative concept of “Conserve and Earn” was recognized by UNESCO and given “World Heritage” recognition. These structures did not suffer during Gorkha Earthquake.

Some of the cultural heritage monuments restored with International assistance suffered severe damages and totally collapsed (See Figure 5). Apparently, earthquake resistance was not in their agenda.

Bhimsen Temple, Lalitpur; Nautalle Durbar, Basantapur; Digutaleju, Lalitpur.
Figure 5. Heritage monuments restored with international assistance damaged during Gorkha Earthquake

Similarly, there are a few instances where local authority intervention damaged structures of cultural heritage post Gorkha Earthquake (Figure 6). Temporary timber struts were erected without any purpose and without the authority’s knowledge and without consultation with local community. The struts were removed again without any information nor evaluation of required strength or caution. This shows lack of ownership at the Government level and lack of consultation with the professional and local community. The world-famous Krishna temple of Lalitpur was damaged by the municipality’s unthinking intervention with erection of timber struts immediately after the quake, causing considerable damage to the temple. Note the damage to ancient inscription on the stone.

Figure 6. Krishna Mandir at Patan, damaged with post-earthquake protection efforts

7. BUILDINGS CODES UPDATE AND PEER REVIEW

The lessons from the earthquake clearly indicate that the building damages are largely dependent on appropriate use of the building codes, quality of construction, proper operation and maintenance, monitoring occupancy change and location. The use of building code itself is a complex process requiring considerable time for design of building based on the code requirements and inelastic design based on computer modeling. The Building owners hardly understand the complexities of time consuming seismic resistant design. More complex is the situation in Nepal where the need for following other international codes is paramount since Nepal Building Code in itself is inadequate and incomplete (Box 1). There is a dare need to update the Nepal Building Code (UNDP/ERRRP: NEP/07/010, 2009) to make it independent of other codes or reduce it to a guideline to help choose better codes. More important is the lack of prevision of a mechanism for Inspection and Code Enforcement (ICE). The lack of prevision for peer review of design, construction and assurance of public safety is indeed very detrimental to serious professionalism.

Box 1: Nepal Building Code Deficiency

Nepal Building Code is divided into four sections: Part 1) State-of-the-Art Buildings, Part 2) Professionally Engineered Buildings, 3) Non-Engineered Buildings (Mandatory Rule of Thumb), and 4) Rural Construction. The code is divided into 22 parts and the seismic design method is specified in NBC 105.

In the preface, NBC 105 has included IS 4326 - 1993 Code of Practice for Earthquake Resistant Design and Construction of Buildings as related code. There is a marked difference between these two codes with various values of the seismic parameters and giving different results. This anomaly has confused most of the practicing engineers and NBC is practically not used. Other factor affecting the use of NBC is the non-accessibility of International software as SAP, ETAB and STAAD Pro which do not recognize NBC.

During Gorkha Earthquake, a lot of buildings designed under NBC 105 Part MRT (Non-engineered Buildings) were damaged. The part of the code is considered inadequate in terms of structural safety and need to be replaced with standard designs for ready use. This part of the code is most misused by the municipality registered designers through copy and paste without a care for details or applicability without giving design considerations and not verified for its acceptability.

The return period, as specified by NBC 105, for the onset of damage for a typical building of ordinary importance has been chosen as 50 years. The return period for the strength of buildings has been chosen as 300 years. NBC 105 specified return period may be an under scored value compared to Katsuichiro Goda recommendation (See Box2).

Box 2- Design earthquake model
The Gorkha Earthquake Damage Survey report (Katsuichiro Goda and et el) recommended that a basis for seismic design comprising the PGA estimates with 10% probability of exceedance in 50 years as the design earthquake model for Nepal. IS 1893 has included two categories of Design Earthquakes: 1) 2 percent probability of exceedance in 50 years (Maximum Considered Earthquake - MCE) and 2) 10 percent probability of exceedance in 50 years (Design Basis Earthquake - DBE) with category 1 structures designed for MCE, which is twice that of DBE, whereas structures in category 2, 3 and 4 are designed for DBE for the project site. ATC 40 has specified 3 levels of earthquake ground motions: 1) Serviceability Earthquake (SE) with 50 percent probability of exceedance in 50-year period, 2) Design Earthquake (DE) with 10 % probability of exceedance in 50-year period, and 3) Maximum Earthquake (ME) with 5% probability of exceedance in 50-year period. ATC 40 has related the level of earthquake with the performance level of buildings which is not the case with NBC 105.

Considering the above earthquake design parameters, the level of risks of structures will depend on the choice of building code selected. Hence, the considered level of risk in every project is different and level of earthquake hazard risk in Nepal also becomes heterogeneous depending on the source of funding. In this context, NBC 105 may need updating to reflect the demand of recent earthquake and future probable earthquakes and may need to develop consensus among the leading professionals and academia about the choice of appropriate earthquake design model.
Having said that, it is imperative that the consistency of design principles is not lost and compliance to the building code requirements or application of correct design criteria and analysis is assured. The need for a unified code acceptable at international level has become imperative.
Apart from this, the assurance of use of appropriate code provisions and correctness of its interpretation and compliance is very important to insure consistency and to eliminate any deficiency through peer review of seismic resistant design and Third Party Verification (TPV) of the quality of design and construction.

8. REBUILDING APPROACH

After the donors meet called by the Government in May 2015, the International Community and the country expected that rebuild initiatives would be launched very quickly and the recovery initiatives started. The Government's effort to establish an independent authority met political and legal hurdles and was practically paralyzed. The Government's post-quake instructions, related to 1) restriction on new construction, 2) reduction of interest on bank loans, 3) short term training of fresh engineers and 4) the creation of National Rebuilding Authority, became redundant due to inadequate homework and preparation and hence could not be formally established even after 6 months. Lack of expert consultation led to unilateral decisions and the general government attitude of "making decisions in haste and repent in leisure" was clear.
The well-wishers from all over the world are quite in panic about Nepal loosing precious time, being unable to gear up for post-quake recovery. With no practical guidelines people started repair and recovery without any engineering or government support and many of the buildings started returned to status quo ante.

Strong voices urge Nepal to learn from the experience of other countries (Japan and New Zealand et el) in earthquake recovery by sending fact finding mission for learning lessons in right approach and policy. The New Zealand's approach to post quake recovery through nomination of the Rebuild Team comprising of industry representatives i.e. the government, consultants, contractors, bankers, suppliers and manufacturers, insurance and community was a unique model that helped New Zealand to recover from the 2011 earthquake in a fast track manner with most effective use of cost and time, employment creation and funds recovered from insurance coverage.
Recently, the september 16, 2015 earthquake with magnitude 8.3 Mw in Chile caused only 13 fatalities. Why only 13 fatalities in this earthquake, considered the world's strongest earthquake to date in 2016 while far weaker earthquakes in Haiti and, more recently, in Nepal, killed tens of thousands? The Chileans very proudly report that the resilience of Chile has three dimensions: a) Strong evacuation plans in coordination with international community as the UN humanitarian affairs office and the International Search and Rescue Advisory Group [Insarag], b) Strict building code that demand all new buildings must be able to survive a 9.0-magnitude earthquake: buildings can crack, tilt and even be declared unfit for future use but must not collapse, and c) Strong and sensitive response to the disaster carried out by Ricardo Toro, a former army general, in-charge of Chile's disaster relief agency, ONEMI.
The 24th August 2016 earthquake (6.2-magnitude) in Amatrice, Accumoli and Pescara del Tronto in mountainous central Italy, killing 240 of people and ruining the whole city reminded Barpak, the epicentre of Gorkha earthquake and indicated the need for taking proactive initiatives before earthquake strikes.
Lack of an institutional model for rebuilding, generally dealing with Earthquake Affairs is the prime reason behind the current chaos in rebuilding regime.

9. ASSISTANCE FOR PROTECTING EXISTING BUILDING STOCK

Protecting existing building stock of Nepal with over 5.5 million is a big challenge in itself. There is not a single building, affected by the Gorkha Earthquake, specially the rural brick/stone buildings in mud mortar. The biggest threat to the rest is from a society that views demolition is the best way in three reasons: 1) The building does not belong to them or the owner is from the different community or neighborhood, 2) It is the easiest way to be safe from the risk it is associated irrespective of the actual physical condition, and 3) There are no funds or technical assistance available for detailed damage assessment and to determine the wisdom of demolition or protection through retrofit techniques. Surely, when wisdom fails and the fear-mongers prevail.

The Chandeswhori temple of Lalitpur, Bhaktapur municipality building and Bhisen stambha of Kathmandu.
Figure 9. Existing Building stock waiting for rebuilding

Demolition and reconstruction of 5.5 million houses is not a figure any economy can afford, and surely not Nepal. If the Bhimsen stambha (Dharahar Tower) reconstruction with a price NPR 3 billion were to contest with the recovery of more valuable heritage objects like Chandeswhori temple, among 15,000 heritage monuments country wide, besides the 745 prepared by the Department of Archeology, not counting the recovery of 5.5 million houses. Nepal Reconstruction Authority needs to look seriously at formulating a judicious policy, with a set of priorities that will ensure the recovery of most valuable national assets associated with the daily life of the people.

10.NEED OF AN EARTHQUAKE SAFETY COMMISSION

Earthquake issues and remedies discussed above leads one to a specific need for a permanent responsible institution in-charge of earthquake affairs, acting as an apex national body that will provide leadership, undertake policy reforms and guide all activities in the sector.
Obviously, there is no common approach to earthquake issues dealt at national or regional or state levels. In the context of Nepal, there is clearly no top-level agency responsible for earthquake issues. It is widely felt that an Earthquake Safety Commission may be required for dealing with the vast scope of rebuilding, preparing for next earthquakes, and mobilizing national and international resources. The Commission may be an independent and autonomous body charged with the mandate to deal with all aspects of earthquake including research and studies, development of technology and policy reforms, performance evaluation, development of strategy for the future, review and updating of building codes, bylaws, guidelines and manuals, conducting training and capacity building, and ensuring overall safety including support for total insurance of residence and infrastructure. Dissemination of this information and knowledge to professionals and community leaders helps to upgrade local community capacities for creating an Earthquake Resilient Society.

11.CONCLUSIONS

Nepal is a highly earthquake prone area with noted earthquakes of magnitudes 4-5 Mw two times a year, one in summer and one in winter. The Gorkha Earthquake of April 25, 2015 is considered as a grand rehearsal for future potential earthquakes based on the historical frequency. The large energy accumulated in the Himalayan Range, particularly around Kathmandu, could rock the area with a land slip of 10 m, which was not fully released during Gorkha earthquake.
The huge loss of life over 8,900 and loss of property about 600,000 collapsed buildings and 500,000 damaged buildings, though a very sad result, is considered significantly less compared to the previously estimated figures. This is a positive result of efforts made during last 3 decades towards creating Earthquake Safer Cities. At the same time, it is also commonly agreed that pre-earthquake preparation was grossly inadequate.
Nepal's march towards Earthquake resilience carries a lot of challenges. In the wake of the recent earthquake and those sure to come, Nepal needs to rebuild over 800,000 buildings and strengthen other existing 5.5 million buildings of adobe construction. Apparently, there is no effective technology to restore, rebuild and strengthen the existing adobe construction. At the same time, updating of the building code and its strict inspection and enforcement would help to ensure an Earthquake Resilient Society in terms of assessment, planning, implementation in a timely manner. The rebuilding initiatives already have been delayed by 16 months. It has disappointed the whole world and the devastated people. But the Government is still not in moving. This is a very pathetic situation, aggravated by the economic embargo at Nepal-India border of september 2015 further delaying the rebuilding and overall progress. The country is slowly going back to the same status of vulnerability as it was before the earthquake.

There are several models of recovery and rebuilding from earthquake disaster. Gujarat, Haiti, Chile and Christchurch are recent models. The Chile model has very strong search and rescue plan, strict building codes that demand for no collapse design, and sensitivity towards Earthquake Disasters. Christchurch model mobilized resources within the country with formulation of a strong and dedicated rebuild team based on non-profit job distribution. Probably, Nepal need to combine and blend together a suitable rebuild course based on world experience.

Creation of Earthquake Resilient Societies and traditional settlements require advanced preparation in the form of: a) Overall Plan for rebuilding and recovery of lost assets, b) strengthening of existing buildings and structures including vulnerability assessment, data base of buildings and infrastructure, technical assistance for damage assessment and design for strengthening, c) implementation of strong building codes and enforcement plan, and d) sensitization towards earthquake disaster. These tasks need meticulous planning, setting priorities, developing tools to enhance access to expertise, building capacity, mobilizing resources, and verifying compliance with standards, along with plans for new construction, strengthening and retrofitting of existing buildings.

The planning, design and effective implementation of earthquake resilience plans require an effective and responsive agency that can take leadership and guide the stakeholders to take delivery of services required for earthquake resilient societies. Two dedicated institutions are in high demand, if the country is to prepare for the next earthquake: 1) Earthquake Safety Commission, and 2) National Building Council to take charge of building code update. The process of institutional building in the post-earthquake rebuilding course of Nepal appears to be straining for release as the initiatives are yet to be recognized and put in place. The steps to be taken, consistent with agreed priorities could be summarized as follows:

- Mobilizing fact finding missions to various countries for learning lessons from previous devastating earthquakes including the Italian Earthquake of September 2016 and identifying effective rebuilding approach,
- Taking initiatives for updating of building codes and defining inspection and code enforcement (ICE) procedures including third party peer review or verification (TPV) including remodeling of the unacceptable design of non-engineered buildings,
- Developing support mechanism for appropriate technology to address local demand for recovery and rebuild of lost buildings and strengthen existing buildings in adobe construction,
- Developing consultation mechanism for addressing professional and community concerns,
- Establishing historical ownership and local community rights on heritage settlements and monuments,
- Providing priority to conservation of cultural heritage monuments and settlements, and economic value return for recovery and rebuild products for sustainability,
- Establishing incentives and motivation packages including reduced interest rates for bank loans, eliminate Government and municipal taxes on rebuild activities and seismic strengthening of properties, and
- Taking policy reforms in updating building bylaws, building act, evacuation and rescue plan, and earthquake hazard insurance.

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