A Szilikátipari Tudományos Egyesület lapja

építőanyag

Journal of Silicate Based and Composite Materials

A TARTALOMBÓL:

- Experiments on the buckling behaviour of glass columns. Part 1.
- Comparison of the inherent variability in rebound hammer tests performed with different testing instruments
- Simple basic model for concrete and its application 2. Factors that influence compressive strength and drying shrinkage
- Lead (II) and zinc (II) ions removal capacity of coarse limestone and rhyolite tuff from aqueous solutions
- Performance of waste glass powder (WGP) supplementary cementitious material (SCM) – Workability and compressive strength

2013/3

THE ROLE OF CEMENT IN THE 2050 LOW CARBON ECONOMY

European cement industry presents five routes to a low carbon future

On 25 September 2013, CEMBUREAU unveiled its vision of what the cement industry could potentially achieve on the road to a low carbon economy of the future. This project has focused on what the sector sees as potentially contributing towards achieving a low carbon economy and the input and opinion of key external stakeholders has been of invaluable use during the project.

The project lays out five parallel routes which can each contribute to lowering emissions related to cement production, as well as concrete production. These have been divided into two groups. The first 3 routes, which cover resource efficiency, energy efficiency and carbon sequestration and reuse, have been quantified for the purpose of this roadmap as they fall under the sector's control. The final two routes (product efficiency and downstream) look at how cement and concrete can contribute to a low carbon society. Nevertheless, potential savings from the two routes outlined do not relate directly to cement manufacturing, so were not included.

Koen Coppenholle, CEMBUREAU Chief Executive noted that "The cement and concrete industry can play a crucial role in helping Europe achieve its goals, since its vision sits well with European requirements and strategic objectives on employment, innovation, education, social inclusion and climate & energy." Nevertheless, he also highlighted the fact that the cement industry cannot do this alone: "We are part of Europe's industrial landscape and depend on other industries, governments and players to be able to deliver on specific parts of this roadmap."

The full roadmap is available from here: http://lowcarboneconomy.cembureau.eu

CEMBUREAU THE EUROPEAN CEMENT ASSOCIATION

építőanyag

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Experiments on the buckling behaviour of glass columns. Part 1.

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Abstract

Supporting structures can be transparent nowadays due to the development of glass strengthening procedures. The building glass as a versatile building material supports architectural design due to its transparency. The paper focuses on load-bearing glass columns and also on the design, the load bearing capacity and the stability issues of fins. International and Hungarian case studies demonstrate the possible use of cross-sections, layers and supporting structures of glass columns [1]. Laboratory experiments were carried out at the BME, Department of Construction Materials and Engineering Geology on buckling of glass columns. More than 60 specimens where loaded until fracture. The load and deformations (buckling, surface deformations) were measured. Based on the experimental results, the critical force was determined and the fracture and stability processes were illustrated by force-deflection diagrams. The results were analysed with the calculation procedures in the focus of the international literature (results are presented separately in the 2nd part of the present paper series).

Keywords: glass column, buckling, load bearing glass, stability, transparency

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1. Glass columns in structural hierarchy

Glass columns belong to the primary structural elements in the structural hierarchy of load bearing glasses *(Fig. 1)*. Glass columns support the secondary and the tertiary elements, which structural elements transfer the load to the primary structural elements that carry the load [1, 2, 3]. The fracture of glass columns used in primary structural elements can cause stability problems in a building, therefore, researchers need to focus more on load bearing and stability questions.

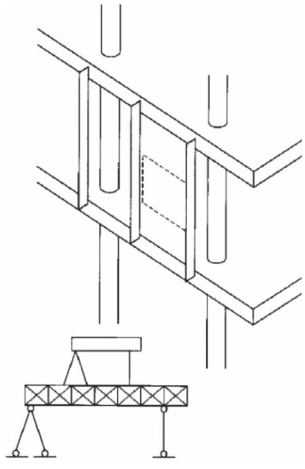

Fig. 1. Hierarchy of structural elements [3, 4, 5]
1. ábra Tartószerkezetek hierarchiája [3, 4, 5]

Glass is used nowadays as a load bearing material due to its transparency, and usually is called the material of the third millennium. With the development of glass strengthening methods, glass has become a frequently used building material in load bearing structures as well [4].

Further investigations are required especially in those areas, where glass is used as a load bearing element. Glass is a brittle material and for a long time its brittleness was a well-known property besides its transparency.

With the development of glass strengthening methods, in the last few years glass began to also be a load bearing material for engineers, which raises several questions. Glass used in columns meet different requirements (to carry loads with limited deformations as well as to be aesthetic), although the structural design of load bearing glass structures is not standardised yet in Hungary.

2. Cross-section of glass columns

During the design of glass columns, engineers have to take into account beside standardised loads – due to the brittle behaviour of glass – special impact loads or non-standardised loads as well, e.g.: impacts that are originated from special concentrated loads: effect of soft-impact e.g. from people or hard-impact e.g. by falling objects. Therefore, it is preferred to carefully select the appropriate location of glass columns inside a building especially when it is used in public areas.

2.1 Cross-section types

Laminated safety glass should be used in load bearing glass columns: at least three layers of heat strengthened glass (HSG) and/or fully tempered glass (FTG) or combination of them is required. The thickness of the interlayer foil should be at least 0.76 mm (type of the interlayer material can be EVA or PVB).

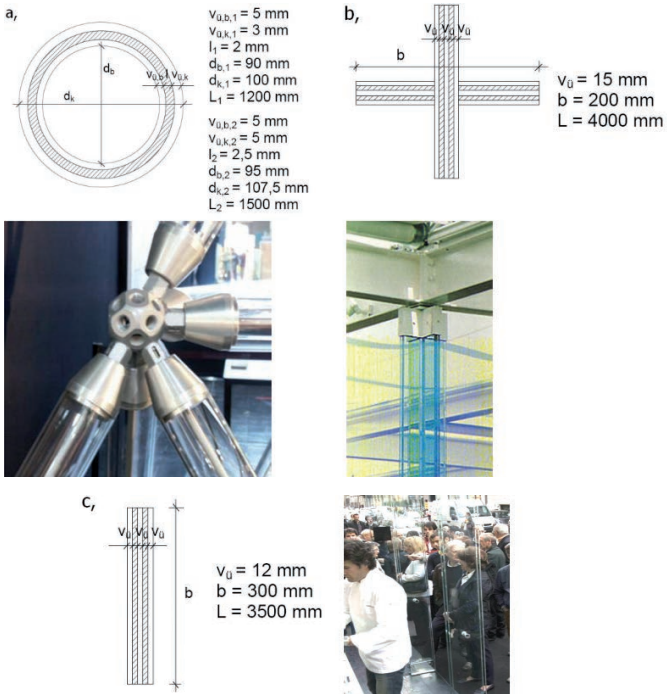

Fig. 2. Types of cross-sections [1, 6, 7]
2. ábra Keresztmetszet típusok [1, 6, 7]

The interlayer material serves two purposes: *(1)* to keep glass splinters in place during the fracture process to reduce the risk of injury and *(2)* to increase residual load bearing capacity.

Different shapes of cross-sections are used in glass columns *(see Fig. 2)* that can be distinguished as:

- Simple cross-sections: cross-section consisted of plane glass layers; circle shaped glass layers.
- Compound cross-sections: cross-section consisted of plane glass layers – square or cross shaped

2.2 Single and multi-storey glass columns

Glass columns can be designed as single or multi-storey structural elements. The type of the supporting structure depends on the height of the glass column. Supporting method can be:

- Glass columns fixed in the region of their lower and upper edges in so called "steel shoe" supporting element. In this case the buckling behaviour should be analysed.
- Suspended method to reduce the effect of buckling. This type of support is preferred to be used in multi-storey façades, where the glass columns are mainly supported independently from the intermediate slabs. In this case the stresses in the region of the bore holes in the glass should be analysed.

2.3 Coupling elements in multi-storey glass columns

Nowadays, glass columns with more than 4 m height are designed in a safe way *(Fig. 3* to *Fig. 5)*, however over 6 m height, coupling elements should be placed.

In general, these coupling elements are constructed with the preparation of bore holes, with the use of screws and steel plates and damping materials. The EN 12150-1:2000 standard determines the requirements on spacing of bore holes in glass.

In recent laboratory experiments, researchers focus on glued glass coupling elements, with the use of overlapping glass layers in laminated glasses.

Main properties of suspended glass columns:

- Construction of glass façade with significant height is possible;
- The self weight and loads of the glazing of the façade are carried mainly by the upper coupling element of the glass column;
- Safety glass consisted of tempered glass layers should be used due to the high stress concentration in the bore hole regions;
- In the case of locations where earthquake with higher magnitude can occur, the glass columns should be suspended.

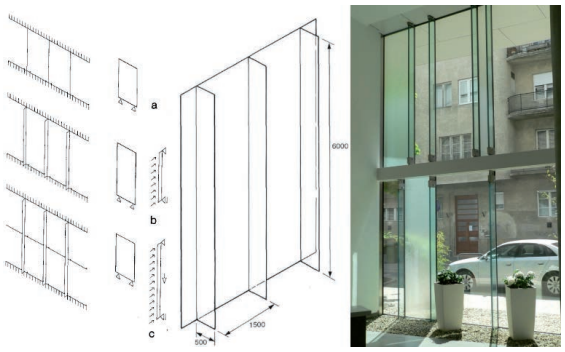

Fig. 3. Single and multi-storey glass columns [1, 5]; Budapest, Víziváros Business centre, Glass columns of Residence 1 building (structural design: Dr. Kinga Nehme)
3. ábra Egy, ill. több szint magas üveg lizénák [1, 5]; Budapest, Vízivárosi irodaházak, Residence 1 épület üveg lizénái (statikus tervező: Dr. Nehme Kinga)

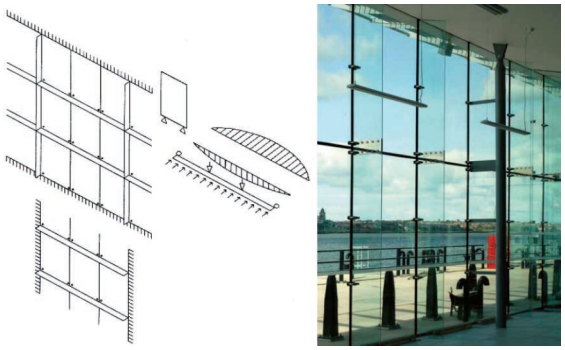

Fig. 4. Multi-storey glass columns with use of Pilkington Planar™ coupling system [1, 8]: Cruise Liner Ferry Terminal, Liverpool, UK
4. ábra Több szint magas üveg lizénák Pilkington Planar™ rögzítéssel [1, 8]: Cruise Liner Ferry Terminal, Liverpool, UK

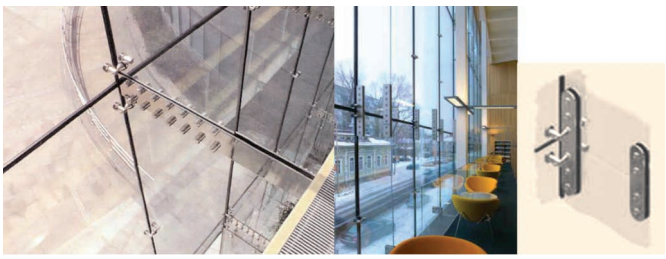

Fig. 5. Spacing of bore holes in vertical and horizontal directions (Library of Turku, Finland) [8]
5. ábra Vízszintes és függőleges furatlyuk kiosztás (Turku könyvtár, Finnország) [8]

3. Laboratory experiments

3.1 Test parameters

Laboratory experiments were carried out to study the buckling behaviour of single and laminated glass columns at the Department of Construction Materials and Engineering Geology, BME. The specimens were tested with use of *INSTRON 5989* universal testing machine. All glass specimens were loaded in compression by concentrated load by variable specimen heights and a constant nominal width of 80 mm. The buckling behaviour and the fracture process were recorded by high-speed digital camera.

Single layer float glass, single layer heat-strengthened glass and laminated glass consisted of both float and heat-strengthened glass layers were tested. Although single layer glass and float glass are usually not used in load bearing glass columns, the effect of heat-strengthening on the buckling behaviour can be studied and can be compared with existing calculation methods in this way. The geometry of test specimens (height, thickness, width) was chosen on the basis of experiences with existing glass columns in buildings in international and Hungarian references.

Test parameters of glass specimens were the followings:

Constants: test arrangement, the type of support; width of glass (80 mm); interlayer material (EVA foil with thickness of 0.38 mm); edgework; temperature (+23 ± 5 °C).

Variables: type of glass layers: HSG/ non heat-treated float; height of specimens: 1000 mm; 920 mm; 840 mm; number of glass layers and the thickness of specimens: single layer: 8 mm; 12 mm, laminated: 2×4 mm; 2×6 mm; 8+4 mm, laminated: 3×4 mm; The rate of loading: 0.5 mm/min; 1 mm/min.

Support: Height of fixing: 95 mm; rubber plate (Shore A 80) was used between the steel supports and the glass.

Simplified designation is used to distinguish the studied specimens; e.g. *H_2(4.4)_2_920_0.5,*

where:

- H, F: Type of glass: H – HSG; F – non heat-treated float glass;
- 2(4.4): Number of glass layers e.g.: 2×4 mm laminated glass;
- 2: The number of specimen;
- 920: Nominal height of specimen [mm];
- 0.5: Rate of loading [mm/min].

3.2 Experimental procedure

The load and vertical displacement of the upper cross-head of the *INSTRON 5989* universal testing machine were continuously measured with *Bluehill* software during the tests of each specimen. At three different heights, the buckling displacement (horizontal displacement) of all specimens were continuously measured with *HBM* displacement transducers during the tests. Strains at centre point on the surface of the glass panels were measured with *HBM LY11-10/120* type strain gauges. The tests were carried out at room temperature (+23 ± 5 °C). At least three specimens were tested at each testing combination. The specimens were loaded until fracture. Laminated specimens were loaded until all glass layers were fractured. In total, 64 specimens were tested. The specimens were mounted as shown in *Fig. 6.*

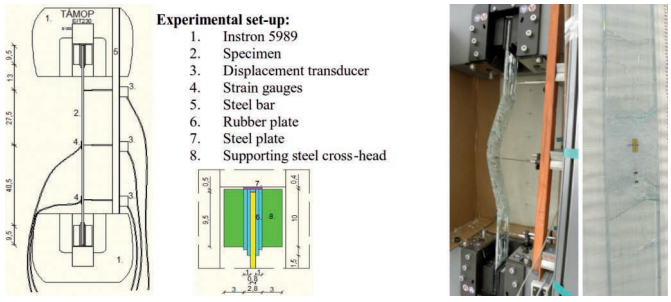

Fig. 6. Test set-up, fractured specimen and strain gauges
6. ábra Terhelési elrendezés, eltört próbatest és nyúlásmérő bélyegek

3.3 Experimental results

Loading force vs. displacement diagrams were prepared for the laboratory experimental results. *Fig. 7* indicates the loading force vs. horizontal displacement in the mid-section of a specimen. *Fig. 8* indicates the loading force vs. vertical displacements. In both *Figs. 7* and *8*, three different stages can be distinguished in the buckling behaviour of the glass columns.

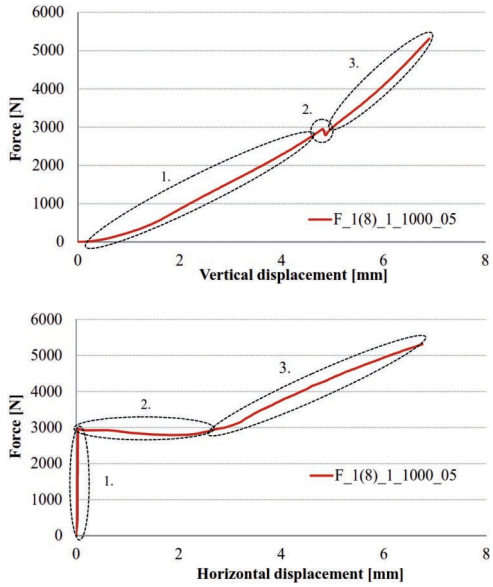

Fig. 7. a) Force vs. vertical displacement b) Force vs. horizontal displacement in the case of single float glass layer with thickness of 8 mm and height of 1000 mm; Stages of buckling behaviour of a glass column
7. ábra a) Terhelő erő és függőleges elmozdulás összefüggése b) Terhelő erő és vízszintes elmozdulás összefüggése egyrétegű float, 8 mm vastag 1000 mm magas üvegek esetén. Üveg oszlop kihajlási alakváltozási szakaszai

In the 1st Stage, the elastic deformation of the damping material (rubber plates) influences the vertical and horizontal displacements and no buckling occur (first stable stage). The 2nd Stage is a short term stage which indicates a geometrical instable condition (in which direction the buckling will occur) and the specimen loses its former stability (bound phenomenon, instability). In the 3rd Stage, both the vertical and the horizontal displacement increase until the fracture of the glass (second stable stage).

Fig. 8 indicates the force vs. vertical displacement curves of single and laminated glass specimens with total thickness of 12 mm. To study the effect of the number of glass layers on the buckling behaviour, single layer glass specimens with thickness

of 12 mm and laminated glass specimens consisted of 2×6 mm or 3×4 mm layers were tested as well. The critical load was found to be reduced with the increase of the number of glass layers. In the 1st Stage, the glass specimens behave similarly, but significant difference can be observed in the 3rd Stage. Before the fracture of the specimen, the force decreases with the increase of number of the glass layers in the case of glass columns consisted of laminated HSG glass layers and with a total thickness of 12 mm. In the case of laminated glasses, the horizontal deformations and the load bearing capacity are influenced by the shear modulus of the interlayer material, therefore the force in the 3rd Stage decreases.

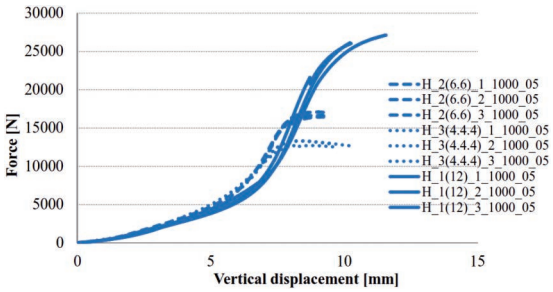

Fig. 8. Force and vertical displacement of HSG single and laminated glass specimens with total nominal thickness of 12 mm and height of 1000 mm

8. ábra Terhelő erő és függőleges elmozdulás, azonos névleges 12 mm vastagságú 1000 m magas, hőkezelt üvegekből felépülő oszlopok esetén

Fig. 9 indicates the force vs. horizontal displacement curves of laminated glass specimens consisted of 2×4 mm HSG glass layers with 1000 mm, 920 mm or 840 mm nominal heights. The critical load and the 3rd Stage was found to be reduced with the increase of the height of glass columns.

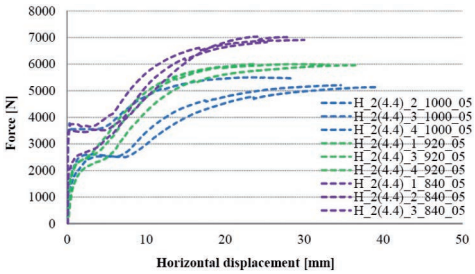

Fig. 9. Force vs. horizontal displacement of laminated glasses consisted of 2×4 mm HSG glass layers with 1000 mm, 920 mm as well as 840 mm nominal heights

9. ábra Terhelő erő és keresztirányú elmozdulás összefüggése 2×4 mm vastag hőkezelt, laminált üvegekből felépülő, 1000 mm, 920 mm, 840 mm névleges magasságú oszlopok esetén

Fig. 10 indicates the comparison in the buckling behaviour of laminated glass columns with the same height but consisted of non heat-treated float glass or HSG glass layers. In the 1st Stage, the glass specimens behave similarly, the 2nd Stage (bound phenomenon, instability) occurs at lower load levels in the case of float glasses, but significant difference can be observed in the 3rd Stage. The 3rd Stage lasted longer time in the case of HSG glass layers with increasing deformations and the force decreased before fracture of the specimen.

The buckling behaviour of laminated glass columns with the same height of 1000 mm and total nominal thickness of 12 mm, consisted of 6+6 mm or 8+4 mm HSG glass layers are compared in *Fig. 11.* No significant difference in the buckling behaviour was observed by applying different thicknesses of glass layers but keeping the same nominal total thickness.

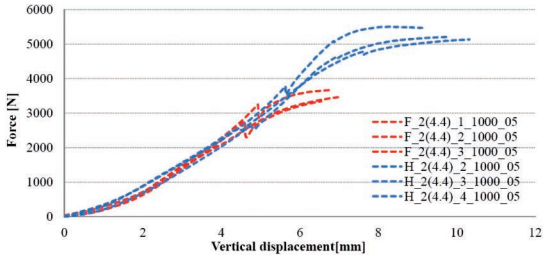

Fig. 10. Force vs. vertical displacement of laminated glasses consisted of 2×4 mm HSG or float glass layers with height of 1000 mm

10. ábra Terhelő erő és függőleges elmozdulás összefüggése 2×4 mm vastag hőkezelt, laminált üvegekből felépülő, 1000 mm, 920 mm, 840 mm névleges magasságú oszlopok esetén

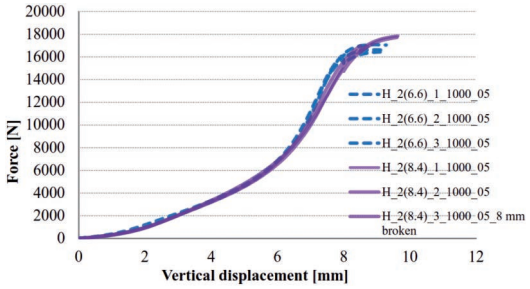

Fig. 11. Force vs. vertical displacement of laminated specimens consisted of 6+6 mm or 8+4 mm HSG glass layers with height of 1000 mm

11. ábra Terhelő erő és függőleges elmozdulás összefüggése (6+6 mm, 8+4 mm) vastag hőkezelt, laminált üvegekből felépülő, 1000 mm magas névleges magasságú oszlopok esetén

3.4 Conclusions

The following conclusions can be drawn for the presented experimental tests:

- Three different stages can be distinguished in the buckling behaviour of glass columns.
- The buckling behaviour is not affected by the loading rate in the case of loading rate of 0.5 mm/min or 1 mm/min.
- The critical buckling load is reduced with the increase of the number of glass layers.
- The allowed buckling load during structural design calculations is suggested to be the maximum load of the 1st Stage (stable stage) reduced with safety factors.
- The 2nd Stage in the buckling behaviour is mainly influenced by the type of the supporting structure (fixed/pinned) and the stiffness of the glass columns.
- In the case of laminated glasses, the horizontal deformations and the load bearing capacity are influenced by the shear modulus of the interlayer material, therefore the force in the 3rd Stage decreases.

Authors have quantitatively summarized the critical load (N_{cr}) of the tested glass columns in *Table 1.* In the case of equal nominal thickness monolithic or laminated glass specimens, the critical load of laminated glass specimens is reduced with 25 to 40 % compared to the monolithic (single) glass specimens. In the case of laminated glass that consists of three glass layers, the reduction can exceed 50 %.

Height [mm]	Type of glass	Critical load (Ncr)									
		Total thickness: 8 mm			Total thickness: 12 mm						
		Single layer	Laminated glass		Single layer	Laminated glass					
		8	4.4		12	6.6		8.4		4.4.4	
		[N]	[N]	%	[N]	[N]	%	[N]	%	[N]	%
1000	Float	5672	3490	62	19803	14575	74	-	-	13425	68
	HSG	7506	5278	70	26420	16698	63	17495	66	12684	48
920	HSG	8784	5989	68	-	-	-	-	-	-	-
840	HSG	10207	6919	68	-	-	-	-	-	-	-

Table 1. Critical load of glass specimens based on the experiments
1. táblázat Kritikus teher a kísérletek alapján

4. Future work

Authors are going to present the existing calculation methods of the critical load of glass columns, and are going to compare the results of the laboratory experiments and theoretical calculations in a separate paper in *Építőanyag – Journal of Silicate Based and Composite Materials.*

5. Acknowledgements

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Üvegoszlopok kihajlásának laboratóriumi vizsgálata. 1. rész.

Az üveg erősítési eljárások fejlődésének köszönhetően ma már a tartószerkezetek is transzparensek lehetnek. Az építési üveg, mint sokoldalú építőanyag átlátszóságának köszönhetően lehetővé teszi az építészek törekvéseinek megvalósítását. Cikkünkben a teherhordó üvegek témakörén belül, az üvegoszlopok, lizénák kialakítási és teherbírási, stabilitási kérdéseivel foglalkozunk. Külföldi és hazai esettanulmányokkal bemutatjuk az üvegoszlopok keresztmetszeti, rétegrendi, megtámasztási és kialakítási lehetőségeit [1].
A BME Építőanyagok és Mérnökgeológia Tanszék laboratóriumában kísérleti úton vizsgáltuk az üveg oszlopok kihajlását. Több mint 60 db próbatestet tönkremenetelig terheltünk. Mértük a terhelő erőt és az alakváltozásokat (kihajlás, felületi alakváltozások). Kísérleti eredményeink alapján meghatároztuk a kritikus erőt, erő-alakváltozás diagramokkal szemléltettük a tönkremeneteli és stabilitási folyamatokat. Eredményeink tükrében elemeztük a nemzetközi irodalomban fellelhető számítási eljárásokat (melyeket a cikksorozatunk következő részében ismertetünk).
Kulcsszavak: üveg oszlop, kihajlás, teherbíró üveg, stabilitás, átlátszóság

XXVIII. Téglás Napok

LACZKÓ László

A Szilikátipari Tudományos Egyesület (SZTE) és a Magyar Téglás Szövetség (MATÉSZ) közös szervezésében 2013-ban 28. alkalommal került megrendezésre a Téglás Napok.
A rendezvények a balatonvilágosi Hotel Frida Family*** adott otthont.

2013. október 10-én 11 órától a MATÉSZ rendkívüli közgyűlésére került sor, majd a konferenciát **Kiss Róbert** (az SZTE Tégla és Cserép Szakosztályának elnöke), valamint **Kató Aladár** (a Magyar Téglás Szövetség elnöke) nyitotta meg.

A nap folytatásaként a hazai tégla- és cserépipart foglalkoztató technológiai illetve épületenergetikai kérdésekről, illetve a hazai építészetünk történetéről hallottunk rendkívül érdekes, hasznos és színvonalas előadásokat.

Elsőként **dr. Matuz Géza**, az ÉMI Építésügyi Minőségellenőrző Innovációs Nonprofit Kft. vezérigazgató-helyettese tartotta meg előadását, melynek témája a nemzeti épületenergetikai stratégia (NÉeS) volt. Az előadó ismertette, hogy e stratégia céljai között az épületek állapotának felmérése, majd az épületek felújításának lehetőségei is szerepelnek. A NÉeS fő célja a felújításokat követően a primer energia felhasználásának csökkentése, a megújuló energia-források arányának növelése és végső soron a CO_2 emisszió csökkentése.

Az ÉMI Nonprofit Kft. november 6-án A Nemzeti Épületenergetikai Stratégia bemutatása címmel rendez konferenciát, melyről a www.emi.hu honlapon találnak bővebb információt az érdeklődők.

Miguel Moix, a spanyol Beralmar Technologic, S.A. export felelőse „Petrolkoksz használatának elterjedése fő üzemanyagként Közép-Európában és a Balkán térségében" címmel tartotta meg előadását. A Beralmar a tégla- és cserépipari szárítók, kemencék és komplett üzemek tervezésével foglalkozik, a vállalat termékeinek alkalmazási területe a szárítási- és égetési technológia, illetve ezek automatizálása. Miguel Moix előadásában hangsúlyozta a szilárd tüzelőanyagok tégla- és cserépipari felhasználásának technológiai és gazdasági előnyeit. Példaként egy 2013. júniusában Boszniában üzembe helyezett berendezést mutatott be az előadó. A prezentációt az érdeklődők megtalálják a www.szte.org.hu internetes oldalon.

„A mikronizált (porlasztott) petrolkoksz felhasználásának előnyei" című előadásában a Garcia-Munte Energia Hungary Kft. tevékenységi körét mutatta be **Urzica Olivér.** A vállalat bemutatása mellett az előadó beszámolt a hazánkban 2013. júliusában indult beruházásról is. A mikronizált petrolkoksz és a petrolkoksz összehasonlítását is tartalmazta az előadás, valamint megismerhettük a mikronizált petrolkoksz felhasználásával járó gazdasági előnyöket is. Az előadáshoz kapcsolódó prezentáció a www.szte.org.hu internetes oldalon teljes terjedelmében megtalálható.

Az előadássorozat zárasaként a BME Magasépítési Tanszékének oktatója, **dr. Déry Attila** Ybl-díjas építészmérnök „Az azonosulás kérdései a magyar építészetben" című kultúrtörténeti összeállítását mutatta be. Az előadás teljes anyaga megtalálható a www.szte.org.hu internetes oldalon.

A XXVIII. Téglás Napok október 10-i előadássorozatát vacsora és baráti beszélgetés követte a Hotel Frida Family*** éttermében.

A rendezvény második napjának (okt. 11.) szakmai programja a Wienerberger Téglaipari Zrt. Balatonszentgyörgyi Tégla- és Gerendagyárának látogatása volt. Itt **Horváth László** gyárvezető fogadta a konferencia résztvevőit, majd egy rövid tájékoztató után megtekintettük a téglagyártás technológiáját.

A gyárlátogatás után a balatonszentgyörgyi Gulya Csárdában zárult a két napos konferencia.

A szakmai konferencia résztvevői: Baranya-Tégla Kft., Berényi Téglaipari Kft., Budai Tégla Zrt.; Castolin Zrt., Creaton Hungary Kft., Körös-Ökotrend Kft., Leier Hungária Kft., Magyar Téglás Szövetség (MATÉSZ); Nagykanizsa Téglagyár Kft., Pápateszéri Téglaipari Kft., QMC Ipari Kerámia Kkt., SZIKKTI Labor Kft., Szilikátipari Tudományos Egyesület (SZTE), Tondach Magyarország Kft., Wienerberger Téglaipari Zrt.

A konferencia résztvevőinek nevében köszönet illeti a Magyar Téglás Szövetségből (MATÉSZ) **Szerdahelyi Ildikót** és a Szilikátipari Tudományos Egyesületből (SZTE) **dr. Antal Józsefnét**, akik precíz szervező munkája, gondossága és tapasztalata nagymértékben hozzájárult a konferencia színvonalához.

Comparison of the inherent variability in rebound hammer tests performed with different testing instruments

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Abstract
The rebound hammer test of concrete can provide an alternative to drilled core tests for estimating the in-situ compressive strength of concrete. The variability of concrete strength is recognised during the strength assessment and the design strength of concrete is specified as its characteristic compressive strength by taking the variability of the strength of concrete into account. In the present paper, particular topics of the inherent variability of the rebound hammer tests are discussed. Different types of Schmidt hammers are compared in this sense. The precision of the original N-type Schmidt hammer is demonstrated to be superior to original L-type or Silver Schmidt N-type hammers. Observations are confirmed by normality tests. Results imply the need for further research. Measurement errors made by the operators of the rebound hammers are analysed. Differences among the repeatability and reproducibility are demonstrated. As a closing remark the need of revealing the connection between the variability of compressive strength and rebound index, as well as the need of the spatial variability analysis of rebound index are highlighted.
Keywords: concrete, rebound hardness, Schmidt hammer, inherent variability, repeatability, reproducibility

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1. Introduction

The variability of concrete strength is recognised during the strength assessment and the design strength of concrete (as characteristic compressive strength) is specified by taking the variability of the strength of concrete into account.

During in-situ testing, the most significant characteristic of the non-destructive tests is that the compressive strength of the concrete is not measured directly in a structure.

Variability can be considered in different ways by different statistical parameters, however, two levels of variability can be attributed to the rebound hammer test: 1) the *inherent variability* corresponding to *one test location* and being influenced by the measurement uncertainties (operator and testing device) and the local inhomogeneity of the concrete, and 2) the *spatial variability* that indicates the measure of the dissimilarity of the inherent variability for *different test locations*. In the present paper, particular topics of the inherent variability of the rebound hammer tests are discussed.

2. Method

The rebound hammer appeared in the 1950's and the rebound hammer test is the most widespread method for the surface hardness testing of concrete.

The rebound hammer (i.e. a tailored spring impact hammer; *Fig. 1.*) was developed in Basel, Switzerland by Ernst *Schmidt* [1]. The hardness testing method of Shore [2] was adopted in the device, and the measure of surface hardness is the *rebound index*. The method became popular very fast as the rebound index can be read directly on the scale of the device and no measurements on the concrete surface are needed [3]. The original idea and design of the device was further developed in 1952 (using one impact spring instead of two) resulted in simpler use [4,5]. In 1954 *Proceq SA* was founded and has been producing the original Schmidt rebound hammers since then, without any significant change in the operation of the instrument [6]. One of the latest developments of the rebound hammers was finalized in November 2007, since the Silver Schmidt hammers are available [7]. The digitally recording Silver Schmidt hammers recorded formerly the original Schmidt rebound index as well, not only the coefficient of restitution (referred as Q-value) of concrete. From 2011, however, the Silver Schmidt hammers are no more instrumented to record the original Schmidt rebound index. Only the Q-value is measurable.

With this skip, the direct relationship between the two hardness values can not be compared any more, and the long-

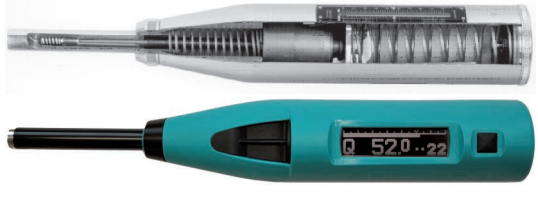

Fig. 1. Original Schmidt rebound hammer and Silver Schmidt rebound hammer
1. ábra Az eredeti Schmidt kalapács és a Silver Schmidt kalapács

term experience with the original rebound hammers, thus the considerable amount of rebound index data can not be used. That can be considered as a drawback from a scientific point of view.

3. Sources of variability and terminology (accuracy, bias, systematic error, random error, repeatability, reproducibility)

During testing, even when none of the experimental parameters is intentionally changed, small changes usually occur both in strength and hardness measurements. These changes are realised as variability among the measured values. Some of the more common (potential) sources of variability during in-situ testing are the operator (performance influences), the testing device (condition and calibration), the environment (temperature, etc.), the test location (heterogeneity of the material). The variability may include systematic components (*bias*) and random components (*error*). The systematic components may be eliminated by correct calibration of the testing device by a reference material.

In statistics, *sampling bias/sampling error* is a deviated sampling when the sample is collected in such a way that some members of the population are less likely to be included than others. Problems with sampling are expected when data collection is entrusted to subjective judgement of human [8]. An example is the systematic error of an observer. *Systematic errors* are very difficult to deal with, because their effects are only observable if they can be removed, but usually they cannot be removed by e.g. increasing the sample size. *Random errors* have zero expected value (scattered about the true value) and tend to have zero mean value when a measurement is repeated. Random errors can be attributed either to the testing device or to the operator. The *accuracy* of statistical information is the degree to which the information correctly describes the phenomena that is intended to be measured [8].

According to the ISO 3534-1 International Standard [9] the *repeatability* is the precision under conditions where independent test results are obtained with the same method on identical test items in the same laboratory by the same operator using the same equipment within short intervals of time. Generally speaking, repeatability shows that how can a person repeat his measurement in the future similarly to that of he did in the past. Therefore, repeatability is the *spread* of measurements made under the repeatability conditions (=measurement variability by the same operator). More specifically, repeatability is a number that is unlikely to be exceeded by the difference between two measurements made under the repeatability conditions.

According to the ISO 3534-1 International Standard [9] the *reproducibility* means the precision under conditions where test results are obtained with the same method on identical test items in different laboratories with operators using different equipment. Generally speaking, reproducibility shows that how can a person reproduce a measurement in the future similarly to that was made by another person in the past. Therefore, reproducibility is the *additional spread* within measurements made under the reproducibility conditions (=additional measurement variability by different operators). More specifically, reproducibility is a number that is unlikely to be exceeded by the difference between two measurements made under reproducibility conditions.

There are different considerations as well. In the nomenclature of e.g. ACI 228.1R-03 Committee Report "In-Place Methods to Estimate Concrete Strength" repeatability is referred as *within-test variation* and reproducibility is referred as *batch-to-batch variation* [10]. Also, repeatability is sometimes called *equipment variation* and reproducibility is called *appraiser variation* [11].

According to the ISO 5725-1 International Standard [12] the *accuracy* of a measurement is described by the *trueness* (=proximity of measured values to the true value) and the *precision* (=either repeatability or reproducibility of the measurement) together. Different accuracies are indicated schematically in *Fig. 2*.

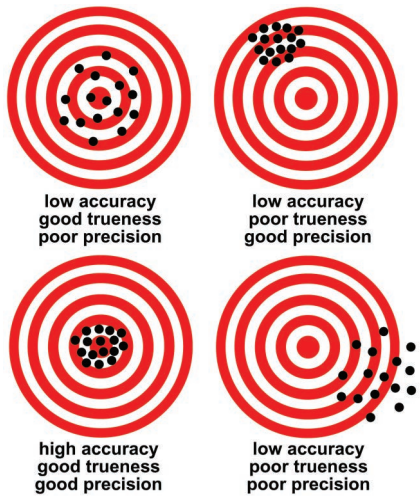

Fig. 2. Schematic representation of accuracy by trueness and precision
2. ábra A pontosság sematikus illusztrációja a valódiság és a precizitás fogalmaival

The uncertainty of the average value of the rebound hammer test reading (either R or Q) depends on three influences: 1) the variability of the strength of concrete in the structure; 2) the repeatability of the rebound hammer test; 3) the number of individual readings [10].

4. Precision statements about the rebound index

The ASTM C 805 International Standard contains precision statements for the rebound index of the rebound hammers [13]. It is given for the precision that the within-test standard deviation of the rebound index is 2.5 units, as "single-specimen, single-operator, machine, day standard deviation". Therefore, the range of ten readings should not exceed 12 units (taking into account a k=4.5 multiplier given in ASTM C 670[14]). Dependence of the within-test standard deviation on the average rebound index is not indicated. Particular literature data support the ASTM C 805 suggestions, e.g. [15].

For the bias of the rebound hammer test no evaluation is given in the ASTM C 805 standard [13]. It is indicated that the rebound index can only be determined in terms of this test method, therefore, the bias can not be evaluated.

5. Precision statements about the compressive strength

The precision of the concrete compressive strength test results is discussed in EN 12390-3 European Standard [16] in terms of repeatability and reproducibility conditions, based on the compressive strength experiments, in which 1) the measurement data include the uncertainties of sampling, specimen preparation, curing and compressive strength test; 2) the measurements were carried out on 100 mm and 150 mm cube specimens. The precision data were determined in the UK in 1987; the concretes were made using an ordinary Portland cement, Thames Valley sand, and Thames Valley coarse aggregates. Hand compaction was used. The ratio of the range and the standard deviation is 2.77 in case of the repeatability, as well as the reproducibility conditions, accordingly the result of two measurements (n=2) was evaluated (*Table 1*).

Specimen type and size	Repeatability conditions		Reproducibility conditions	
	s/f_{cm}, %	r/f_{cm}, %	s/f_{cm}, %	r/f_{cm}, %
100 mm cubes	3.2	9.0	5.4	15.1
150 mm cubes	3.2	9.0	4.7	13.2

Table 1. Precision data for measurements of the compressive strength of hardened concrete, expressed as percentages of the mean of the two cube strengths whose difference is to be compared with repeatability or reproducibility [16]
1. táblázat Megszilárdult beton nyomószilárdsági vizsgálat precizitásának megadása két kocka nyomószilárdság vizsgálata alapján ismételhetőségi és reprodukálhatósági feltételek esetén [16]

It is indicated that the difference between two test results from the same sample by one operator using the same apparatus within the shortest feasible time interval will exceed the repeatability value on average (r/f_{cm}=9 % in case of 150 mm cube) not more than once in 20 cases (5%) in the normal and correct operation of the test method.

It is also indicated that the difference between the test results on the same sample obtained within the shortest feasible time interval by two operators each using their own apparatus will exceed the reproducibility value on average (r/f_{cm}=13.2 % in case of 150 mm cube) not more than once in 20 cases (5%) in the normal and correct operation of the test method.

Further information on precision, and for definitions of the statistical terms used in connection with precision can be found in ISO 5725-1 International Standard [12].

6. Comparison of different types of rebound hammers

6.1 Variability of the different rebound indices

The very robust design of the original Schmidt hammers provide full protection to the mechanical parts by a sturdy metal housing. The calibration and maintenance is simple. If the maintenance of the instrument is regular and its moving parts are kept clean, then failure and erroneous operation can only be expected if the operator is unskilled or not careful and implements a very severe operational error. The original Schmidt hammers record the rebound index (R): the ratio of paths driven by the hammer mass during rebound and before impact; see Eq. (1).

$$R = \frac{x_r}{x_0} \cdot 100 \qquad (1)$$

where x_0 indicates the path driven by the hammer mass before impact, and x_r indicates the path driven by hammer mass after impact, respectively.

The Silver Schmidt hammer – the competitor of the original Schmidt hammer – has some operational differences. The mechanical components of the device are operated, except for a few modifications (e.g. the hammer mass has lower weight than that of the original design), by the same principle as the original instruments. However, in the Silver Schmidt hammer the velocity of the hammer mass is measured by optical measuring transformer. The light is modulated by the grooves on the circumference of the hammer mass and transmitted to a photodiode. The duration of the impact/rebound periods is expressed by the ratio of the velocities. The result of the velocity measurement is not influenced by the gravitation (so the rebound value requires no angular correction). As the hammer mass of the Silver Schmidt hammer is of less weight, it runs the forth and back paths quicker. The weight of the plunger of the device is also smaller than that of the original Schmidt hammer. Therefore, and also due to the different measuring method, the Q value of the Silver Schmidt hammer is greater than the R value of the original Schmidt hammer. The Q-value represents the ratio of kinetic energies of the hammer mass right after and just before the impact (E_0 and E_r, respectively); see Eq. (2).

$$Q = \frac{E_r}{E_0} \cdot 100 = \frac{v_r^2}{v_0^2} \cdot 100 \qquad (2)$$

where v_0 indicates the velocity reached by hammer mass before impact, and v_r indicates the velocity reached by hammer mass after impact, respectively.

The first generation of the Silver Schmidt hammers was implemented to record both R and Q values. Manufacturer *Proceq SA* provided the devices with the conversion curve represented in *Fig. 3* [7]. The second generation of the Silver Schmidt hammers, however, does not record the R values. According to the manufacture's opinion, no relationship is suggested between the Q and the R rebound values [17].

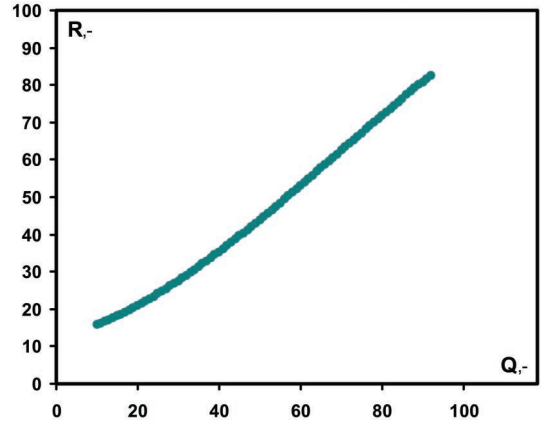

Fig. 3. Relationship between rebound index R and rebound index Q [7]
3. ábra Az R és a Q visszapattanási érték kapcsolata [7]

This is noteworthy for several reasons. On the one hand, previous experience of many years applies to the original R rebound values. Practically no data are available in the technical literature related to the new devices. The standards also formulate guidelines for the original Schmidt hammers. On the other hand, the changes in the operating and measuring principle raise questions concerning the repeatability and reproducibility: does the new device provide similar measurement uncertainty than that of the original Schmidt hammer or does show better or less favourable performance?

In this respect, very limited number of references can be found in the technical literature, and the experiences are not favourable so far.

It has been shown on 10 different natural stones that the necessary sample size to arrive at the same confidence level of the estimation of the sample mean is considerably higher for the Silver Schmidt hammer than is needed for the original Schmidt hammer, regardless the magnitude of the operator observational error [18]. The digital data collection for the Q value (Silver Schmidt hammer) instead of the operator's eye sensory reading for R value (original Schmidt hammer) *does not improve* the precision of the measurement. This clearly indicates that the measurement uncertainties, i.e. the repeatability parameters of Silver Schmidt hammer are less favourable than that of its predecessor. It calls the attention to further analyses.

A comparative study of four different rebound indices was performed by laboratory testing on 11 individual, identical concrete cubes of 150 mm. The testing devices were an L-type original Schmidt hammer, an N-type original Schmidt hammer and a first generation Silver Schmidt hammer capable to record both R values and Q values. *Table 2* summarizes test results. 20 rebound index recordings were taken by each device on each specimen. It can be seen that the highest precision corresponds to the N-type original Schmidt hammer (highest precision means here the lowest range and the lowest standard deviation for the measured values at individual test locations). Lower precision of the L-type original Schmidt hammer and of the Silver Schmidt hammer is due to the lighter hammer masses impacting within both devices and the sensitivity of the electro-optical recording (Silver Schmidt hammer).

It can be observed in *Fig. 4* as well, that the rebound indices can be recorded at a larger uncertainty in case of lower impact energy: the standard deviation and the coefficient of variation of the rebound indices of the L-type original rebound hammer is larger than that of the N-type original rebound hammer. Moreover, it can be realized that the electro-optical recording of the Silver Schmidt hammer involves considerable uncertainty: the uncertainty of the Q value is about 20% larger and the uncertainty of the R value is 50-60% larger than that of the R value provided by the original N-type rebound hammer. *Fig. 4* is based on the average standard deviation and average coefficient of variation of the 11 cubes of the same concrete composition (f_{cm}=64.7 N/mm^2, R_{Nm}=42.13).

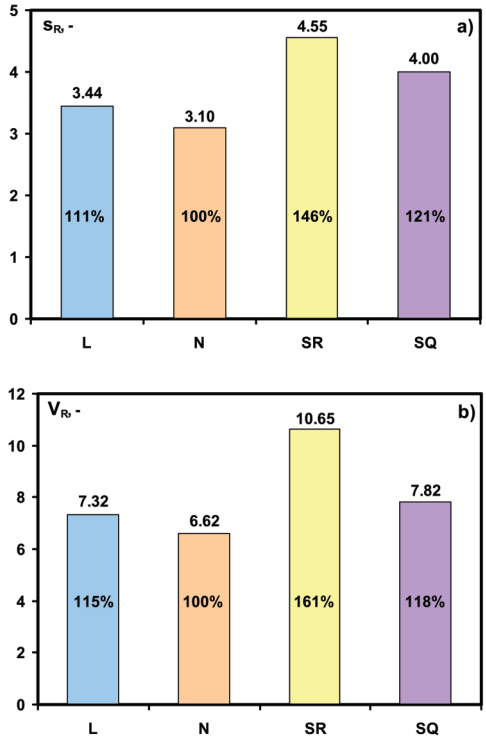

Fig. 4. Comparison of a) the standard deviation and b) the coefficient of variation of different rebound indices
4. ábra Különböző visszapattanási értékek statisztikai jellemzőinek összehasonlítása a) szórás és b) relatív szórás

		specimen										
		1	2	3	4	5	6	7	8	9	10	11
R_L (L original)	mean	45.2	43.4	41.6	45.3	46.4	44.8	44.3	43.1	47.4	47.0	45.2
	stand. dev.	4.7	4.6	4.4	4.6	4.2	5.6	5.0	6.2	3.6	4.4	4.3
	range	15	19	18	15	15	17	14	26	14	13	17
R_N (N original)	mean	47.0	45.6	41.9	46.6	46.3	41.8	45.0	42.1	43.0	46.7	42.2
	stand. dev.	3.0	2.9	4.0	3.4	3.1	3.8	2.0	3.7	2.8	3.7	3.1
	range	11	10	16	12	10	13	7	12	10	14	12
R (Silver)	mean	46.8	41.3	41.5	41.9	42.8	42.9	45.1	41.6	42.0	40.4	40.9
	stand. dev.	6.4	5.1	3.2	4.8	3.0	5.3	5.1	4.4	4.1	3.8	4.3
	range	22	21	12	17	12	20	21	18	18	13	17
Q (Silver)	mean	50.5	48.1	46.2	47.4	45.3	48.3	47.4	47.5	46.6	52.3	48.5
	stand. dev.	3.1	4.0	3.5	3.5	3.0	2.9	3.8	4.1	3.7	4.6	3.8
	range	11	18	13	13	11	11	17	17	15	16	15

Table 2. Statistical characteristics of rebound indices obtained by different types of rebound hammers [19]
2. táblázat Különböző visszapattanási értékek statisztikai jellemzőinek összehasonlítása [19]

The observations may also serve to explain in parts why the manufacturer has terminated the recording of the R value in the second generation of the Silver Schmidt hammers. It is acceptable that such unreliable performance is not used for hardness parameter determination that could provide large measurement uncertainty to the user. Inferior precision of the Q value reading compared to that of the original Schmidt rebound index is still need to be analysed further.

Another open issue is that in-situ measurements of the Q value can not be related to the earlier experiences of more decades and, therefore, the user performing the strength estimation could rely only on the strength estimation relationships of the manufacturer company (which is not a scientific research institute) and which strength estimation curves are formulated on the basis of limited scientific background and limited in-situ experience. The attention of material testing engineers should be called to these limits, because the strength estimation of a structural concrete of unknown composition is a challenge even with the original Schmidt hammers of good repeatability characteristics (it should be also emphasised here that the EN 13791:2007 European Standard [20] does not propose the use of any non-destructive method without drilled core samples).

The consequences of the increase in the inherent measurement uncertainty of the Silver Schmidt hammers by their design and development are topics for future research.

6.2 Normality of the different rebound indices

The normality tests are used in mathematical statistics to study if a data set has normal distribution or not. The normality tests can be useful for the rebound hammer tests since normality is a frequent presumption in statistical procedures. There are about 40 normality tests available in the technical literature [21], however, the most common normality test procedures of statistical analyses are the *Shapiro-Wilk test*, the *Kolmogorov-Smirnov test*, the *Anderson-Darling test* and the *Lilliefors test*. It is demonstrated in the technical literature that the Shapiro-Wilk test is the most powerful normality test from the above four [22]. An analysis is introduced here for the rebound hammer test based on the Shapiro-Wilk normality test.

It can be assumed for the rebound hammer test that the rebound index reading sets of separate test locations are independent and identically distributed (i.i.d.) random variables. It is expected that the probability distribution of the rebound index does not change by location within the same concrete structure and the independent test locations can be considered to be mutually independent. The central limit theorem can be considered acceptable for the rebound hammer test; i.e. the probability distribution of the sum (or average) of the rebound index reading sets of independent test locations (each with finite mean and finite variance) approaches a normal distribution if sufficiently large number of the i.i.d. random variables is available.

It can be demonstrated by the running of the Shapiro-Wilk normality test, whether the probability distribution of the rebound index reading sets of individual test locations can be described by normal distribution or not. The Shapiro-Wilk normality test can be considered as a practical application of the central limit theorem for the rebound index reading sets of individual test locations and, therefore, may be a good indicator for the precision of the rebound hammer test.

During the comparison, four different rebound indices were compared by the laboratory testing of 11 individual, identical concrete cubes of 150 mm (with average compressive strength of f_{cm} = 64.7 MPa) detailed above. The testing devices were an L-type original Schmidt hammer, an N-type original Schmidt hammer and a first generation Silver Schmidt hammer capable to record both R values and Q values. The Shapiro-Wilk normality test was run and the W statistic was calculated for 1, 2, 3, …, 11 rebound index reading sets combined.

Fig. 5 summarizes the values of the W statistic as a function of increasing number of specimens. It is demonstrated that values of W statistic approaches the fastest to unity for the N-type original Schmidt hammer. It implies its superior precision. The tendencies are similar for the L-type original Schmidt hammer, but the W statistic shows lower values. Results are controversial in the case of the Silver Schmidt hammer. Tendency of the values for the W statistic seem to decrease rather than increase, which contradicts probability theory and apparently indicates that the central limit theorem does not apply. The observed behaviour highlights again the concerns of the electro-optical data collection. The results, however, confirm the long term advantageous experiences with the N-type original Schmidt hammers (see e.g. [18] as well).

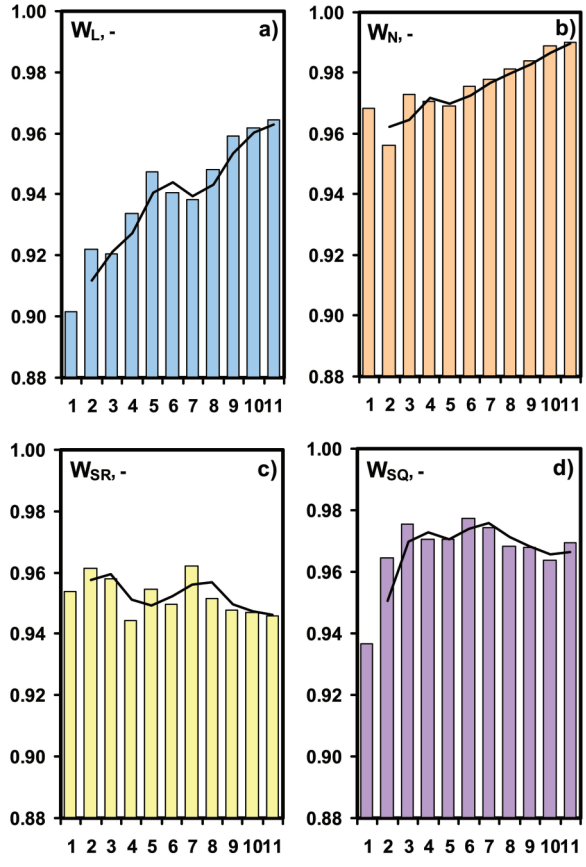

Fig. 5. Comparison of the Shapiro-Wilk test W statistic of different rebound indices a) L-type original Schmidt hammer, b) N-type original Schmidt hammer c) Silver Schmidt hammer R values d) Silver Schmidt hammer Q values

5. ábra A Shapiro-Wilk normalitás vizsgálat W statisztikája különböző visszapattanási értékekre vonatkozóan a) L-típusú Schmidt kalapács b) N-típusú Schmidt kalapács c) Silver Schmidt kalapács R-érték d) Silver Schmidt kalapács Q-érték

6.3. Reproducibility of the rebound indices

The concept of reproducibility of the rebound method includes the error produced by the operator as well. In this regard, three operators contributed in a reproducibility analysis presented here. Rebound hammer tests were carried out on 2×3=6 standard cube specimens of two different strength classes. The average concrete compressive strengths were $f_{cm,1}$=65.8 N/mm² (MIX1) and $f_{cm,2}$=89.7 N/mm² (MIX2). All the three operators recorded 20-20 individual readings on each specimen with both N-type and L-type original Schmidt hammer. The results were evaluated by the type of concrete and per the operator, separately. It was realized – concerning the repeatability of measurements – that the standard deviation and the coefficient of variation of the rebound index is smaller in the case of the higher strength concrete (with higher average rebound index), independently of the operator. It was also observable that none of the three operators made performance errors during the measurements: the average rebound values recorded on the same concrete, by all three operators were almost identical; trueness of the measurements can be considered good in all cases. However, consequences can not be drawn for the reproducibility based on the average rebound indices. As a possible approximation, the standard deviations of the measurements and particular statistical parameters of the standard deviations were chosen for the comparison of uncertainty attributed to the operators. It was considered that an operator is accurate, if the standard deviation of the *mean** of the measurement is small, and an operator is precise if the standard deviation of the *standard deviation** of the measurement is small (* it is noted that *mean* is defined here as the arithmetic mean of individual rebound indices at a test location and *standard deviation* is defined here as the standard deviation of individual rebound indices at a test location). Strictly speaking, this concept does not meet the definition of accuracy and precision, but it is acceptable as an analogy if the average rebound values are considered as the true value, and the variation around this true value is analyzed. Thus, if the standard deviation of the individual measurements of an operator is small at a test location, it means that the readings are implemented in the narrow vicinity of the true value. And if the standard deviation of the standard deviation of the individual measurements of an operator is small considering multiple test locations, it means that the limit of the reading range around the true value varies in narrow vicinity. Evaluation of the test results are presented in *Fig. 6*.

Although the differences are not significant at all, but based on the above principles, the most accurate and most precise operator (operator 2), an accurate and less precise operator (operator 1), and the least accurate and least precise operator (operator 3) can be assigned. During the reproducibility studies it should always be evaluated separately that to which reasons the differences observable at the operators can be attributed and how the differences can be overcome by e.g. refining the testing skill. This is important by two reasons in case of the Schmidt hammer tests (even with a correctly calibrated device, i.e. with favourable repeatability parameters) because both *operational error* (incorrect handling and operation of the device) and *observational error* (incorrect reading on the scale of the device) can be implemented by the operators.

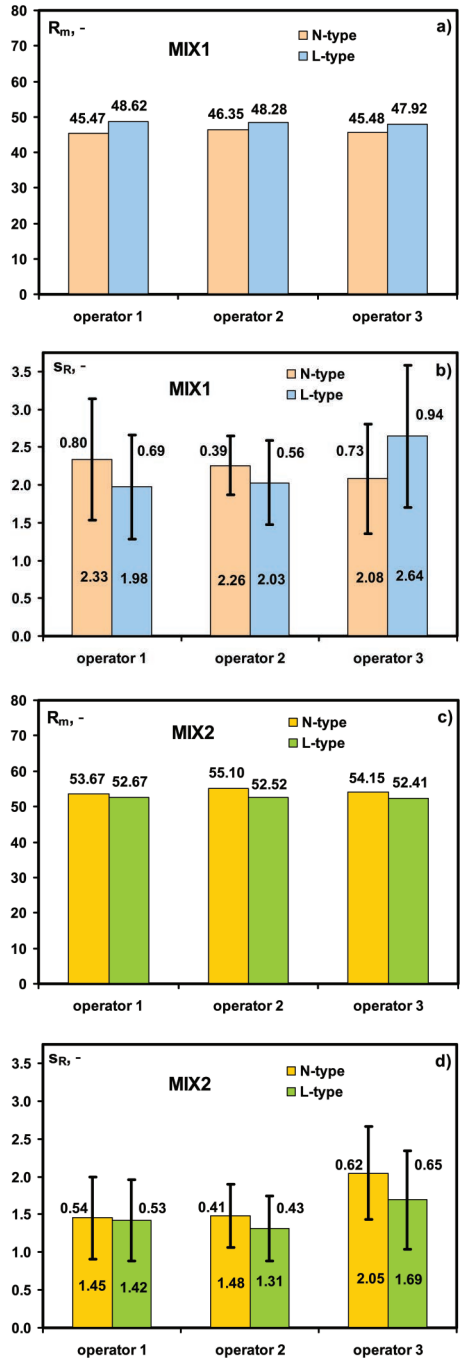

Fig. 6. Repeatability and reproducibility analysis of the rebound hammer test
6. ábra Ismételhetőségi és reprodukálhatósági jellemzők Schmidt kalapácsos vizsgálatra vonatkozóan

7. Need for spatial variability analyses

There is considerable interest, from a practical point of view, on a possible connection of the repeatability of in-situ measurements (e.g. the rebound hammer tests) and that of the compressive strength tests: namely, it is an open question if the coefficient of variation of the rebound index (V_R) could be an acceptable estimate of the coefficient of variation of concrete compressive strength (V_f). Reliability analyses need the value of V_f, however, it is time consuming and expensive to establish its value in a practical situation. Rebound hammer test, on the other hand, is easy to perform and may provide a far less time consuming and expensive solution in a practical situation.

According to the EN 12390-3 European Standard [16], repeatability of strength is represented as the normalised range. It is demonstrated elsewhere [19] that the normalised range defined in EN 12390-3 could not be acceptably used as a simple adaptation to the rebound indices of test locations for the estimation of the repeatability of compressive strength. For the same purpose, the spatial variation of the rebound index between multiple test locations can be considered as more promising rather than the coefficient of variation corresponding to the individual test locations (e.g. by the adaptation of variograms to depict spatial variability in geostatistics [23,24]). Results in this field are presented in a separate paper [25].

8. Conclusions

The variability of concrete strength should be recognised during the in-situ strength assessment. The rebound hammer is the most widespread method for the surface hardness testing of concrete and at the same time one of the most widespread NDT method for concrete strength estimation.

In the present paper, particular topics of the inherent variability of the rebound hammer tests are discussed. The inherent variability of the rebound hammer test corresponds to a test location and is influenced by the measurement uncertainties (operator and testing device) and the local inhomogeneity of the concrete.

Standardised precision statements are presented in terms of repeatability and reproducibility conditions both for compressive strength test and rebound hammer test.

The original L-type and N-type Schmidt hammers record the rebound index (R): the ratio of paths driven by the hammer mass during rebound and before impact. The new Silver Schmidt hammers, however, are implemented to record the Q values (coefficient of restitution). Very limited number of references can be found in the technical literature about Silver Schmidt hammer test results and according to the manufacturer there is no direct relationship between the original R rebound value and the new Q rebound value, so the relationships used earlier can not be applied for strength estimation anymore.

It can be demonstrated based on a comparative study of concrete specimens that the highest precision corresponds to the N-type original Schmidt hammer and lower precision of the L-type original Schmidt hammer and of the Silver Schmidt hammer is realised; supposedly due to the lighter hammer masses (both devices) and the sensitivity of the electro-optical recording (Silver Schmidt hammer). The uncertainty of the Q value is about 20% larger and the uncertainty of the R value is 50-60% larger than that of the R value provided by the original N-type rebound hammer. The results confirm the long term advantageous experiences with the N-type original Schmidt hammers and the consequences of the increase in the inherent measurement uncertainty of the Silver Schmidt hammers that need future research.

A simplified reproducibility analysis of the rebound hammer test was introduced by involving three operators. Results demonstrated that the standard deviations of the measurements and particular statistical parameters of the standard deviations can be acceptable for the comparison of the uncertainty attributed different operators.

9. Acknowledgements

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http://dx.doi.org/10.14382/epitoanyag-jsbcm.2013.14

Különböző típusú Schmidt-kalapácsokkal végzett keménységvizsgálatok mérőhelyen belüli változékonyságának összehasonlítása

Szerkezeti betonok helyszíni roncsolásmentes szilárdságbecslésének egyik eszköze a Schmidt kalapács, amely bizonyos esetekben alternatív megoldást jelenthet a kifúrt magminták vizsgálata helyett. A szerkezeti beton nyomószilárdságának változékonyságát az anyagvizsgálat illetve a szerkezettervezés során a nyomószilárdság karakterisztikus értékében vesszük figyelembe. Cikkünkben a Schmidt kalapácsos mérésből származó egyes mérési bizonytalansági paramétereket vizsgálunk, többféle Schmidt kalapács összehasonlításával. Rámutatunk, hogy az eredeti konstrukció alapján készült N-típusú Schmidt kalapácsok precizitása jobb, mint az L-típusú Schmidt kalapácsoké vagy a Silver Schmidt kalapácsoké. Elemezzük a Schmidt kalapácsos vizsgálat ismételhetőségi és reprodukálhatósági paramétereit is.
Kulcsszavak: beton, felületi keménység, Schmidt kalapács, mérési bizonytalanság, ismételhetőség, reprodukálhatóság

Együttműködési megállapodás

a Budapesti Műszaki és Gazdaságtudományi Egyetem Építőmérnöki Kar, Építőanyagok és Mérnökgeológia Tanszék és a Magyar Cementipari Szövetség között

Az oktatás és az iparvállalatok nemzetközivé válása, az európai integráció, az Európai Unió tagállamaiban egységes piaci, jogi környezetre való törekvés következtében az egyes tagállamokban a felsőoktatás és a termelő vállalatok egyaránt érdekeltek a megszerzett tudás megosztásában, és ismereteik gyarapításában. Ez a közös szemlélet a gyártástechnológiákon túl különösen az új fejlesztésű építőanyagok, alapanyagok, azok felhasználása, beépítése, annak technológiája, a környezetvédelmi és jogi környezet, a szabványos vizsgálatok és a kutatás-fejlesztés területén kölcsönös előnyt biztosítanak a képzési intézmény és az iparág számára.

Ezzel összefüggésben a BME Építőmérnöki Kar, Építőanyagok és Mérnökgeológia Tanszék és a Magyar Cementipari Szövetség (MCSZ) megállapodnak abban, hogy a kutatás-fejlesztés valamint a felsőfokú szakemberképzés, szakmérnök képzés, és az ahhoz kapcsolódó tevékenységek területén együttműködésbe kezdenek. Az együttműködés kölcsönös bizalmon és egymás szaktudásának és szakértelmének elismerésén alapszik. A közös kezdeményezést a Felek lépésről lépésre fejlesztik, és alakulását figyelemmel kísérik. Az együttműködésben az oktatási intézmény részéről a Tanszék oktatói, a Szövetség részéről tagjainak szakemberei vesznek részt. Az együttműködés létrehozásával további cél még az oktatási rendszerből kikerülő, frissen végzett szakemberek versenyképes, naprakész, az éppen alkalmazott technikai színvonalnak, szabályozási környezetnek megfelelő tudáshoz és tapasztalathoz segítése.

Az Együttműködési megállapodás alapját képezi kutatás-fejlesztési feladatokban való együttműködéseknek, pályázatokban való közös részvételeknek, valamint az ipari szakembereknek a bekapcsolódását különféle oktatási formákba. Mindezek szolgálják a Felek hatékonyabb és eredményesebb működését, valamint az oktatásban részt vevő hallgatók naprakész tudáshoz, míg az iparvállalatok felkészült szakemberekhez való jutását.

Budapest, 2013. július 17.

Dr. Balázs L. György	Dr. Dunai László	Szarkándi János
tanszékvezető	dékán	elnök
BME Építőmérnöki Kar		MCSz

Cooperation agreement

between the Budapest University of Technology and Economics, Faculty of Civil Engineering, Department of Construction Materials and Engineering Geology and the Hungarian Cement Association

Higher education institutes and industrial partners in the countries of the European Union are interested in sharing the available knowledge with each other and the further progress in science and education; due to the European integration, the international cooperation and the integrated legal and market regulations within the EU. This common viewpoint provides advantages to both parties in R+D through the development of novel construction materials and production technologies, basic materials, installation and use of materials, environmental protection techniques, standard testing methods and the harmonisation of the legal environment.

The BME Faculty of Civil Engineering, Department of Construction Materials and Engineering Geology and the Hungarian Cement Association (MCSZ) agrees to start a cooperative work in the fields of R+D, top level professional education, post-graduate professional education and connected areas. The cooperation is based on the common trust and the recognition of the technical knowledge and practical skills of each other. The common initiative is going to be monitored and developed by the two participating parties together. Participating members of the contribution are university lecturers and researchers from BME and professionals from the members of MCSZ. Distinguished aim of the cooperation is to accommodate graduate students receiving their BSc or MSc degree with up to date knowledge and experience to become competitive, high standard professionals ready to use their knowledge and skills in the actual technical and legal environment of the building construction industry.

The treaty is expected to be a basis of further R+D cooperation, participation in grant applications and can provide possibility for professionals in practice to join to the different levels of higher education. The efforts can serve a more effective operation of the participating parties, to have graduate students accommodated with up to date knowledge and to provide industrial partners with skilled professionals.

Budapest, 17 July 2013

Dr. György L. Balázs	*Dr. László Dunai*	*János Szarkándi*
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Simple basic model for concrete and its application

2. Factors that influence compressive strength and drying shrinkage

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Abstract

By introducing dimensionless concrete composition content indicators, the structural composition of concrete mixes can be graphically described, opening the way for analysing the effects which influence the performance properties of fresh and hardened concrete mixes using an approach that differs slightly from what has gone before. The present paper deals with the continuation of the observations and the analysis of the results. By an analysis of factors that have influence on the compressive strength, it is possible to make further progress towards an explanation for the two problems mentioned in the literature (summarised in the first part of the present series of papers), which have never so far been fully understood. The analysis of drying shrinkage deformation tests of small-scale prisms prepared during the laboratory experiments is also presented, and the relationships between the concrete composition content indicators and the drying shrinkage deformations in hardened concretes are highlighted. An interesting parallel is revealed in the effects that influence compressive strength and those that influence drying shrinkage deformation, and this provides an opportunity to estimate drying shrinkage deformation in advance from the concrete composition content indicators.

Keywords: concrete technology, concrete mix design, concrete composition content indicators

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Chemical engineer (University of Veszprém, 1981). Active in the construction industry since 1984, first as a research engineer, and later as head of the laboratory at ARÉV, which was at the time the leading construction company in Székesfehérvár. From 2000, as a private consultant, he prepared enterprises for the implementation and operation of a number of QA systems (ISO 9001, ISO 14001, ISO/IEC 17025). From 2007-2013 he was a part-time engineering inspector at the state-owned ÉMI (Non-Profit Limited Liability Company for Quality Control and Innovation in Building). With the support of ÉMI, he carried out a research project, originally in conjunction with the Augusztin Concrete Manufacturing Company (Zamárdi), to establish the correlations between concrete compositions and the performance indicators of the set concrete, by recording observations of concrete mixes during their manufacture at the mixing plant. In 2011, he was invited by the Hungarian Institute for Transport Sciences to participate in research into low-shrinkage floor concrete compositions.

1. Comparison of manufacturing plant observations and laboratory experiments

The series of plant observations are reported in the first part of the paper [1]. The cooperation received support from ÉMI Nonprofit Ltd., which also began an internal *series of laboratory experiments*, with the aim of carrying out further investigations of the effects of concrete composition content indicators. The number of observations *suitable for analysis* has currently reached 220 for field observations and 53 for laboratory mixes. The observed concrete composition content indicators are summarised in *Table 5*.

A particular observation or experiment can be considered for inclusion in the analysis if both the *composition* of the concrete mix and the *concrete composition content indicators* are known (p – volumetric ratio of paste in the concrete, x – volumetric ratio of free fluid and paste powder in the paste, χ_c – the volumetric ratio of cement in the paste powder, λ_{AD} – volumetric ratio of admixtures compared to the paste powder, l – volumetric ratio of air in the concrete), and *at least one performance property* of the fresh or hardened concrete has been *measured*.

Measured performance properties during the *industrial* observations can be:

- *consistency* of the fresh concrete (flow table test),
- *compressive strength* of hardened concrete (after 28 days standard curing).

During the *laboratory* experiments (at ÉMI Nonprofit Ltd.), *drying shrinkage* was also tested up to the age one year.

2. Evaluation of compressive strength results

It is assumed that the reader is familiar with the standard procedures for estimating compressive strength in the literature. The most widely known procedures are based on placing the effect of the water-cement ratio (as the *only*

		Content indicators					Other technical data			
		p	x	x_c	λ_{AD}	l	Types of cement taken into account	Tradi-tional w/c	a*	D_{max} mm
ÉMI Nonprofit Ltd. 2010. 53 experiments	min.	0.098	0.565	0.084	0.000	0.000	CEM I 42.5 N, CEM III/B 32.5 N-S	0.202	0.495	4
	max.	1.000	2.633	0.99	0.055	0.202		3.758	0.769	8
	AVG.	0.303	1.432	0.715	0.015	0.040		0.912	0.666	8
	STD.	0.110	0.566	0.275	0.019	0.028		0.757	0.064	1
Augusztin Betongyártó Ltd. 2008–2010. 220 observations	min.	0.153	1.041	0.552	0.000	0.000	CEM I 42.5 N, CEM II A-M (V-LL) 42.5 N, CEM III/A 32.5	0.384	0.652	4
	max.	0.346	3.378	0.985	0.03	0.081		1.218	0.79	32
	AVG.	0.254	1.683	0.834	0.009	0.018		0.685	0.728	24
	STD.	0.023	0.305	0.082	0.010	0.011		0.145	0.019	9

Table 5. Concrete composition content indicators and concrete technology parameters, of analysed mixes from plant observations and from experimental mixes. (:a - the volumetric ratio of aggregates in the concrete mixes)*

5. táblázat Az üzemi megfigyelések és kísérleti keverések során értékelésbe vont keverékek betonösszetételi állapotjelzői és néhány egyéb adatai (a: a betonkeverékben lévő adalékanyag térfogatarányа)*

factor) into mathematical formulations. Regarding methods of estimating strength which are "w/c based", Ujhelyi and Popovics have demonstrated that *besides* the w/c ratio there are *other variables* (e.g. *cement content* or *water content*) which can be incorporated to improve the statistical properties of the estimates [2]. Nevertheless, even these improved estimation methods do not account for the *increase in compressive strength experienced when there is an increase in the content of additions* to concrete mixes with *identical water-cement ratios and identical cement content* [3], which suggests that there may be *additional factors* that influence the compressive strength and have not so far been taken into consideration.

Before turning to the analysis of our own observations, the Feret method is introduced, already over 100 years old, but not frequently used in Hungary. The mathematical formula for the Feret method as published by Ujhelyi [4] is shown in Eq. (29).

$$f_{c,28} = A \cdot \left(\frac{c}{1-a} \right)^2, \text{ where } a \le a_{max} \qquad (29)$$

where $f_{c,28}$ is the expected compressive strength of a standard cube cured for 28 days [N/mm^2],

A is the experimental constant,

c is the *volume* of cement in the concrete,

a is the *volume* of aggregate in the concrete, and

a_{max} is the *volume* of the maximum amount of aggregate that can be compacted in the concrete

Eq. (29) of the Feret equation is *clearly based on volumes*, and therefore compatible with the simple model for concrete mixes, and so it follows that, with some changes to the equation to give us the new Eq. (30), we end up with the concrete composition content indicators which are discussed in the present series of papers:

$$f_{c,28} = A \cdot \left(\frac{\chi_c}{(1+x) \cdot \left(\frac{l}{p} + 1 \right)} \right)^2 \qquad (30)$$

Form (30) of the Feret equation makes possible to *analyse* the effects of particular concrete composition content indicators on strength, and can even be used for estimating the strength of *unsaturated concretes*, although when the air content is extremely high it predicts compressive strengths that are too high to be realistic. Note: concrete is unsaturated if

$$p + a_{max} < 1. \text{ Then: } l = 1 - a - p, \text{ where } a \le a_{max} \qquad (31)$$

The Feret equation may be a good source of ideas for seeking relationships that can be interpreted within even broader limits.

2.1 Dependence of compressive strength on concrete composition content indicators

Taking, as an example, the 98 mixes of the *industrial observations* which were made using CEM I 42.5 N cement, and which also contained limestone powder as an addition, statistical calculations also proved that there was a *very strong* relationship (correlation coefficient R^2=0.8219) between compressive strength and the w/c ratio, and a *medium* relationship (R^2=0.5455) between compressive strength and content indicator x. The correlation between compressive strength and the other concrete composition content indicators – at least when their effects were examined *in isolation* – was quite *weak*, but when the content indicators were considered *together*, even the weakly correlating factors became quite significant.

Performing calculations on the available observations and experimental mixes, we arrive at Eq. (32):

$$\ln f_{c,28} = n_\chi \cdot \ln \; _c + n_p \cdot \ln p - n_x \cdot \ln(1+x) + n_l \cdot \ln(1-l) + \ln A \qquad (32)$$

The same relationship in *multiplication factor exponent* form is Eq. (33):

$$f_{c,28} = A \cdot \frac{\chi_c^{n_\chi} \cdot p^{n_p}}{(1+x)^{n_x}} \cdot (1+l)^{n_l} \qquad (33)$$

where $f_{c,28}$ is the expected compressive strength of a standard cube cured for 28 days [N/mm^2],

A is the experimental constant,

χ_c is the volumetric ratio of cement in the paste powder, n_χ is the exponent of χ_c,

p is the volumetric ratio of paste in the concrete, n_p is the exponent of p,

x is the *volumetric ratio* between the fluid and the paste powder, n_x is the exponent of $(1+x)$,

l the volumetric ratio of air in the concrete, and n_l is the exponent of $(1-l)$.

The parameters of Eq. (33) are summarised in *Table 6*, based on the observations and experiments carried out with different types of cement, in the analytical ranges laid out in *Table 5*.

	A	n_χ	n_p	n_x	n_l	Note
CEM I 42.5 N	295.371	0.434	0.275	1.665	3.0	ÉMI Non-profit Ltd., 2010
CEM III/B 32.5 N-S	360.055	1.027	0.334	2.084	3.0	
CEM I 42.5 N	585.666	1.190	0.272	2.033	4.0	AUGUSZTIN Betongyártó Ltd., 2008–2010
CEM II A-M (V-LL) 42.5 N	342.302	1.711	-0.240	2.355	3.75	
CEM III/A 32.5	862.337	1.289	0.961	1.860	3.0	

Table 6. The parameters of estimation Eqs. (32) and (33) for the different types of cement investigated during experiments at ÉMI in 2010 and observations at Augusztin Betongyártó Ltd. between 2008-2010.

6. táblázat A (32) illetve (33) becslőképlet paraméterei a vizsgált különféle cementfajták esetében az ÉMI 2010. évi vizsgálatai és az Augusztin Betongyártó Kft. 2008-2010. közötti megfigyelései során

For the 98 mixes made from CEM I 42.5 N and containing limestone powder, referred to in section 2.1., the correlation coefficient of strength estimation made using Eq. (32) or the equivalent Eq. (33) was R^2=0.8689, and the individual significance of the concrete composition content indicators can be proved using a *t-test*. Eq. (33) is reminiscent of Eq. (30) of the Feret equation, with the exception that in Eq. (33) the multiplication factors have *independent exponents* and the *l/p* ratio is separated into independent *p* and (1-*l*) factors. Eq. (33) *correlates better with experience* even with high air content, *within the analytical ranges under observation*. Extrapolations can only be made only *hypothetically*.

2.2 Dependence of compressive strength on the *w/c ratio* and *cement content*

Ujhelyi and Popovics demonstrated that introducing an additional variable (such as *c - cement content*) to the w/c ratio resulted in improvements to the statistical properties of w/c based estimates [2]. Taking this view as our starting point, we also recalculated our own results, adding to a new variable of *cement content as mass ratio* (*c/R*), while continuing to calculate with air content as volumetric ratio. For the results of the observations and the experiments, we have found the equation in Eq. (34) to be suitable:

$$\ln f_{c,28} = n_c \cdot \ln\left(\frac{c}{R}\right) - n_w \cdot \ln\left(\frac{w}{c}\right) + n_l \cdot \ln(1-l) + \ln A \qquad (34)$$

where $f_{c,28}$ is the expected compressive strength of a standard cube cured for 28 days [N/mm²],

A is the experimental constant,

c/R is the mass ratio of cement in the concrete (*R*: concrete density), n_c is the coefficient of *c/R*,

w/c is the traditional water-cement ratio, n_w is the coefficient of *w/c*,

l is the volumetric ratio of air in the concrete, and n_l is the coefficient of *l* .

Eq. (34) can also be expressed in the form of a multiplication exponent. With regard to the 98 mixes made with CEM I 42.5 N and containing limestone powder, referred to in section 2.1., the correlation coefficient of the estimate is R^2=0.8668. The *statistical significance* of the factors *c/R* and *w/c* and (1-*l*) can be proved separately using a *t-test*.

The parameters of Eq. (34) are summarised in *Table 7*, based on the observations and experiments carried out with different types of cement, in the analytical ranges laid out in *Table 5*.

	A	n_c	n_w	n_l	Note
CEM I 42.5 N	54.645	0.236	0.587	3.0	ÉMI Non-profit Ltd., 2010
CEM III/B 32.5 N-S	33.630	0.274	0.922	3.0	
CEM I 42.5 N	43.924	0.197	1.104	4.0	AUGUSZTIN Betongyártó Ltd., 2008-2010
CEM II A-M (V-LL) 42.5 N	12.135	-0.173	1.6670	4.6	
CEM III/A 32.5	238.516	0.996	0.521	2.4	

Table 7. The parameters of estimation Eq. (34) for the different types of cement investigated during experiments at ÉMI in 2010 and observations at Augusztin Betongyártó Ltd. between 2008-2010

7. táblázat A (34) becslőképlet paraméterei a vizsgált különféle cementfajták esetében az ÉMI 2010. évi vizsgálatai és az Augusztin Betongyártó Kft. 2008-2010. közötti megfigyelései során

2.3 Comparison of the methods for compressive strength estimation

The two methods for strength estimation employed with regard to the 98 mixes made with CEM I 42.5 N and containing limestone powder, referred to in section 2.1., can be studied in *Fig. 11*. At first look, the two methods used in the range of observations appear to be more or less equivalent.

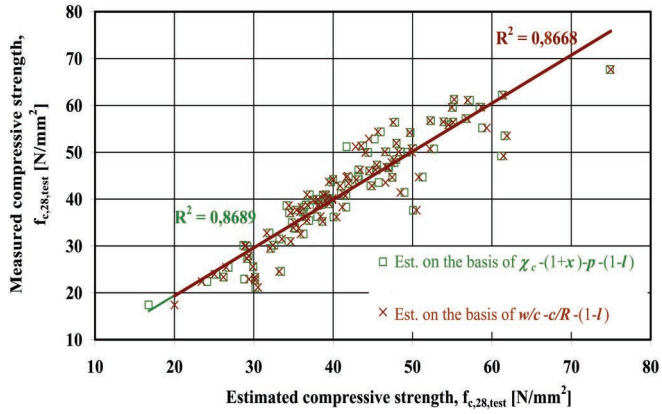

Fig. 11. Comparison of the errors of Eqs. (32) and (34) in estimating compressive strength

11. ábra A nyomószilárdság (32), illetve (34) képletek becslési hibáinak összehasonlítása

We may also compare the two methods by taking a look at the pairs of nomograms on *Figs. 12* and *13*, and *Figs. 14* and *15*. The nomograms were made by processing the results of two types of cement investigated during the ÉMI experiments in 2010, by inserting suitable parameters from *Tables 6* and *7* into Eqs. (32) and (34), and placing the results in the appropriate analytical ranges in *contour* diagrams. Bands of the same colour between the contours indicate identical expected levels of compressive strength, and the scales are identical in the pairs of nomograms.

The first observation is that the *compressive strengths of concretes with CEM III/B 32.5 N-S approach or even exceed the compressive strengths of similar mixes with CEM I 42.5 N, when cement contents are high and water-cement ratios are low.* The nomograms in *Figs. 14* and *15* are particularly eloquent: when w/c=0.2 and c/R=0.22, both cements have an expected compressive strength of ~95 N/mm². This, therefore, provides an explanation for the apparent contradiction mentioned in [5]. The figures reveal that the influences exerted on the strength of cements can be described *by characteristics*, and not be discretely measured specific properties, and these characteristics depend on the concrete composition content indicators of the mixes, which *must not be ignored during mix design*.

The second observation is that the estimates of compressive strength based on *w/c-c/R-(1-l), when cement content is high* and *w/c is low* are progressively less sensitive to the increase in cement content, and extremely sensitive to the *reduction in w/c*. The predicted results in these ranges are unstable. In estimates based on *x-*χ_c*-p-l,* this effect appears much more moderate at the extremes of the range.

The third observation is that the *w/c-c/R-(1-l)*-based estimate of strength for identical w/c and identical cement content always predicts identical strength, which contradicts the results reported in [3], which stated that compressive strength increases as the dosage of additions increases. At the same time, the effect of additions on strength *can be predicted* from estimates *based on x-*χ_c*-p-l,* as we shall prove in section 3.

In our opinion, the use of estimates of compressive strength based on the *single variable* w/c in modern concrete engineering

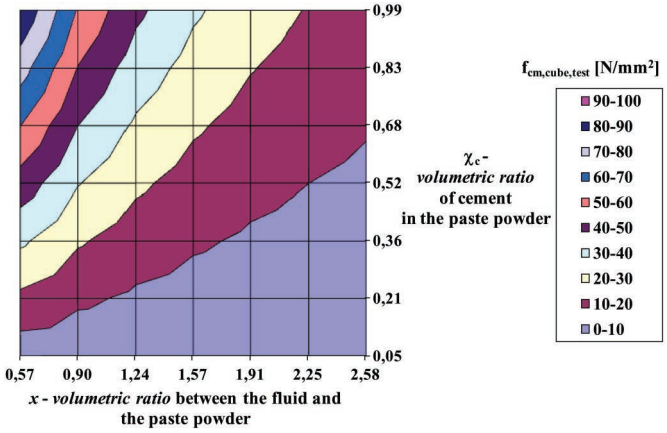

Fig. 12. x-χ_c-p-l-based nomogram for mixes with stone powder and CEM III/B 32.5 N-S, based on experiments at ÉMI in 2010, (p = 0.280; l = 0.010)

12. ábra Az x-χ_c-p-l-alapú becslés nomogramja CEM III/B 32,5 N-S cementtel készült kőlisztes keverékekre, az ÉMI 2010. évi kísérletek feldolgozása alapján, p=0,280 és l=0,010 esetén

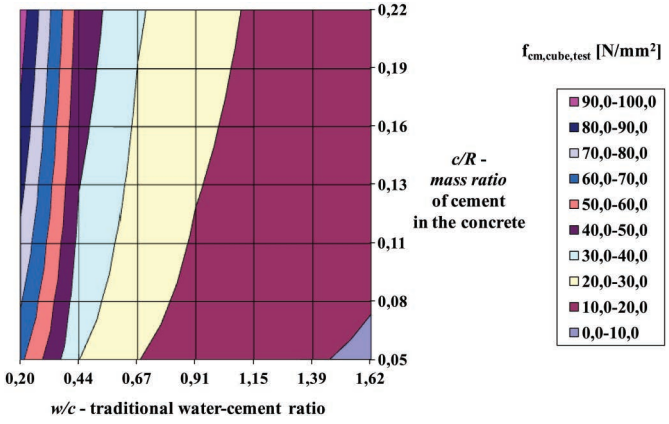

Fig. 14. w/c-c/R-(1-l)-based nomogram for mixes with stone powder and CEM III/B 32.5 N-S, based on experiments at ÉMI in 2010, (l = 0.010)

14. ábra A v/c-c/R-(1-l)-alapú becslés nomogramja CEM III/B 32.5 N-S cementtel készült kőlisztes keverékekre, az ÉMI 2010. évi kísérletek feldolgozása alapján, l=0,010 esetén

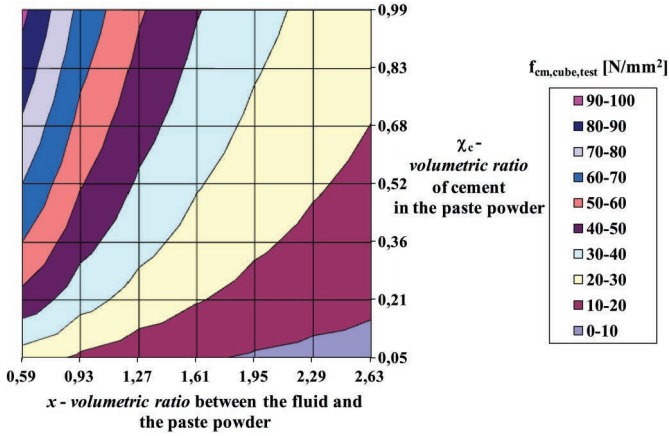

Fig. 13. x-χ_c-p-l-based nomogram for mixes with stone powder and CEM I 42.5 N, based on experiments at ÉMI in 2010, (p = 0.280; l = 0.010)

13. ábra Az x-χ_c-p-l-alapú becslés nomogramja CEM I 42,5 N cementtel készült kőlisztes keverékekre, az ÉMI 2010. évi kísérletek feldolgozása alapján, p=0,280 és l=0,010 esetén

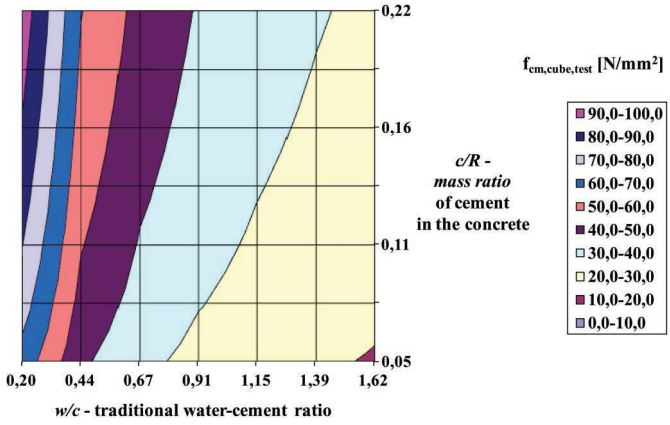

Fig. 15. w/c-c/R-(1-l)-based nomogram for mixes with stone powder and CEM I 42.5 N, based on experiments at ÉMI in 2010, (l = 0.010)

15. ábra A v/c-c/R-(1-l)-alapú becslés nomogramja CEM I 42,5 N cementtel készült kőlisztes keverékekre, az ÉMI 2010. évi kísérletek feldolgozása alapján, l=0,010 esetén

is *of concern*. These methods can be significantly improved by introducing one additional variable (e.g. *cement content*), but there is still a need - disregarding *air content* - to include another variable (e.g. χ_c) in order to describe the influence of the content of additions. This brings us to the point we already reached with the application of concrete composition content indicators in section 2.1.: *in general*, four independent variables are necessary for estimating the compressive strength.

3. The effect of additions on compressive strength

For illustration, let us produce two concrete mixes containing 350 kg/m³ cement, with a water-cement ratio *w/c*=0.5, and containing 1% (v/v) air (*l*=0.01). Mix no. 1 contains no additions (χ_c=0.99), while mix no. 2 contains enough for χ_c=0.55. If we calculate the expected compressive strengths of the two mixes using Eq. (32), on the basis of the parameters acquired from the ÉMI experiments, which are in the first two rows of *Table 6*, then the results are as given in *Table 8*.

Table 8 clearly shows that it is possible to predict, using the *x-χ_c-p-l-based estimation*, the change in the compressive strength expected from an increase in addition content. It can

ser. no.	recipe Material	kg/m³	content indicators χ_c	x	p	l	λ_{AD}	$f_{c,28,\ estimated}$ [N/mm²] CEM I 42.5 N	CEM III/B 32.5 N
	cement	350							
	additions	3							
1.	aggregates	1840	0.990	1.485	0.293	0.010	0.057	44.7	34.4
	water	168							
	admixtures	7							
	cement	350							
	additions	258							
2.	aggregates	1592	0.550	0.825	0.387	0.010	0.031	62.6	39.3
	water	168							
	admixtures	7							
			compressive strength increase					40%	14%

Table 8. Two concrete mixes with a cement dosage of 350 kg/m³ and a water-cement ratio of w/c=0.5, but with different dosages of additions, and their estimated compressive strengths calculated from the concrete composition content indicators, equation (32) and the parameters contained in Table 6 (based on ÉMI experiments, 2010)

8. táblázat 350 kg/m³ cementadagolású, 0,5 víz-cement tényezőjű betonkeverék megvalósítása két különböző kiegészítőanyag-adagolás mellett, és azok becsült szilárdságai a betonösszetételi állapotjelzőkből, a (32) képletből és a 6. táblázat paramétereiből (ÉMI kísérletek, 2010. alapján)

be observed that in mix no. 2 the figure for x fluid-powder volumetric ratio was significantly diminished (from x=1.485 to x=0.825), which exerts a strengthening effect. Nevertheless, there are some types of cement and some ranges of concrete composition content indicators where strength does not show an increase, but rather a stagnation or even a decrease. This example was only shown to highlight the important role played by the *characteristic behaviour* of concrete constituents, which depends not only on the quality of the materials, but also on the concrete composition content indicators.

4. Investigation of deformations during the laboratory experiments

During the *laboratory experiments* at ÉMI Nonprofit Ltd., deformations were also tested by a Graf-Kaufman apparatus. The components of the concrete mixes were the same as those referred to in *Table 5*. Basalt powder was used as an addition. The concrete composition content indicators of the mixes evaluated are laid out in *Table 9*.

Three prismatic specimens were prepared from each concrete mix, with nominal dimensions of 40×40×160 mm, and brass measuring spikes were inserted into the ends while the concrete was still fresh. The prisms were kept for 1 day in moulds covered with a damp cloth, and at the age of 1 day they were removed from the moulds. The initial masses of the prisms were measured, as well as the precise *lengths for comparison* (nominal 160 mm + protruding measuring spikes). The specimens were stored under water until day 7, after which they were stored under laboratory conditions. The temperature and humidity were constantly monitored and recorded. Changes in the mass and length of the prisms were measured at regular intervals, that is at days 1, 7, 14, 21, 28, 56, 112, … , from which the *relative* values for drying and deformation were *calculated*. Changes in mass and dimension were expressed as a percentage, marked for +/- (*negative*: *mass reduction* and *shrinkage*; *positive*: mass increase and expansion).

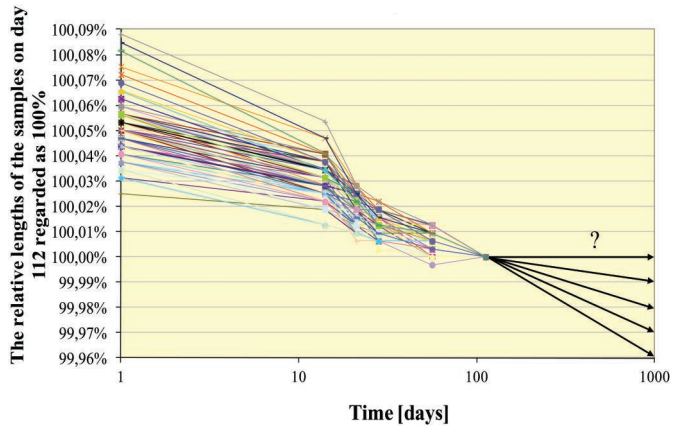

Fig. 16. The deformation process of samples made with CEM I 32.5 N over a logarithmic timescale, with the dimensions of the prisms on day 112 regarded as 100%. The arrows indicate the possible values for deformation projected to day 1000, by continuing the trends. The question mark is there because all projections of this nature need to be validated through real measurements

16. ábra A CEM I 32,5 N cementből készült hasábok alakváltozási folyamata logaritmikus időskálán szemléltetve, ha a hasábok 112 napos méreteit tekintjük 100%-nak. A nyilakkal az eddigi trend folytatódása esetén az 1000 napos korra prognosztizálható alakváltozás lehetséges értékeit jelöljük. A „?" utalás arra, hogy minden prognózist csak valós mérési eredmények hitelesíthetnek

Results are demonstrated here up to the 112 days of age measurements. It can be interesting to know, however, *which stage of the deformation process has the concrete reached by day 112 in its unrestrained deformation process?* A clear answer to the question can only be given when further results are available, so the measurements are continuing. The picture emerging from the measurements can be seen in *Fig. 16*. The diagram shows the deformations measured on prisms made with CEM I 42.5 N at days 1, 14, 21, 28, 56 and 112, but in the diagram the *base for comparison (100%) is the length measured on day 112*, and not on day 1, and the relative lengths of the prisms have been calculated and shown relative to this base. By necessity, the curves all intersect at a single point (100%) on day 112, revealing quite spectacularly the diffuse dynamics of the deformation process that has gone before, and hinting at the possible values of deformation to be expected subsequently,

Types of cement used for deformation tests conducted by ÉMI		Content (structural) indicators					Other parameters and data						
		p	x	x_c	λ_{AD}	l	Traditional w/c	a*	f_a** m^2/m^3	f_z*** m^2/m^3	f_z/f_a	c/R****	D_{max} mm
CEM I 42.5 N 27 experiments	MIN.	0.098	0.690	0.379	0.000	0,000	0.251	0.495	1587	1.186×106	111	4.66%	4
	MAX.	0.472	2.633	0.988	0.049	0.202	1.624	0.769	12168	1.352×106	747	22.26%	8
	AVG.	0.293	1.430	0.762	0.016	0.043	0.665	0.665	7753	1.248×106	186	12.48%	8
	STD.	0.082	0.572	0.214	0.019	0.036	0.315	0.066	1855	5.522×104	121	4.76%	1
CEM III/B 32.5 N-S 25 experiments	MIN.	0.203	0.684	0.084	0,000	0.000	0.252	0.522	3145	1.352×106	119	2.54%	4
	MAX.	0.465	2.583	0.988	0.051	0.064	3.758	0.757	12168	1.442×106	452	21.69%	8
	AVG.	0.294	1.454	0.657	0.014	0.038	1.177	0.668	7990	1.388×106	183	9.97%	8
	STD.	0.072	0.555	0.32	0.019	0.016	0.981	0.063	1458	2.773×104	60	5.67%	1

*Table 9. Concrete composition content indicators of mixes evaluated as part of the deformation tests conducted by ÉMI in 2010 (p: paste ratio in the concrete, x: volumetric ratio of liquid compared to the paste powder, χ_c: volumetric ratio of cement in the paste powder, λ_{AD}: volumetric ratio of admixtures to paste powder, l: air volumetric ratio in the concrete), and other data (*a: volumetric ratio of aggregates in the concrete mix, **f_a: volumetric specific surface area of the aggregates [∅>0.063 mm] calculated using Kausay's method [6], ***f_z: volumetric specific surface area of the paste powder [∅<0.063 mm], ****c/R: the mass ratio of cement in the concrete)*

*9. táblázat Az ÉMI 2010. évi alakváltozási vizsgálatai során értékelésbe vont keverékek betonösszetételi állapotjelzői (p: péparány a betonban, x: folyadék térfogatarányа a pépporhoz képest, χ_c: cement térfogatarányа a pépporban, λ_{AD}: adalékszer térfogatarányа a pépporhoz, l: levegő térfogatarányа a betonban), és egyéb adatai (*a: a betonkeverékben lévő adalékanyag térfogatarányа, **f_a: az adalékanyag [∅>0,063 mm] térfogati fajlagos felülete Kausay eljárása szerint [2] számítva, ***f_z: a péppor [∅<0,063 mm] térfogati fajlagos felülete,****c/R: a cement tömegarányа a betonban)*

on this logarithmic timescale. Based on the illustration, *at least* two thirds of the process of shrinkage that is projected for the 1000-day period has already developed by the day 112, and there is a *suspicion* that *the deformation - all other conditions being equal - could be proportional to the time logarithm* (or to the logarithm raised to the power of *x*), which shall be confirmed, or disconfirmed, by statistical analysis of the results. The analyses of the results gained after 3 years period is just being under processing. The present paper merely deals with the 112 day results.

5. Factors that influence concrete deformation

Detailed literature review is outside the scope of the present paper. According to the traditional view, the kind of concrete deformation that we have tested can be classified as *drying shrinkage*, and *Fig. 17* indicates one representative example from the technical literature [7].

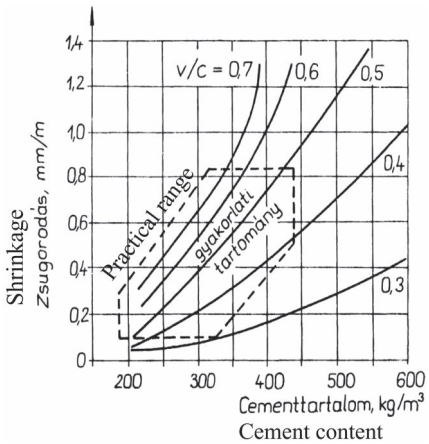

Fig. 17. Depiction of shrinkage with influencing parameters [7]
17. ábra A zsugorodást befolyásoló egyes tényezők hatása [7]

We would like to quote here two statements, the first by János *Ujhelyi* [8]: "Concerning the composition of concrete, *shrinkage increases* when the dosage of cement and water increases, *and the quantity of fine particles in the aggregate increases* (italics added by the author of the present paper). But even more important than these effects are the external conditions [9]: the method and conditions of storage." The second quotation is by Attila *Erdélyi*, [10]: "*The reduction in the w/c ratio is therefore the best method* of moderating drying shrinkage ε_{cds}, - *together with* retention of the *cement content*. If w/c= 0.3-0.35 and c= 450 kg/m^3, then the predicted total drying shrinkage will be 0.3-044 ‰ for time t=∞".

Our own experiments have only dealt with a part of the complete range, and have inevitably investigated the effects of just a few concrete composition materials, but our aim was to use freshly measured factual data to seek confirmation of the statements given in the literature. We were also interested in whether it was possible to demonstrate the effects of concrete composition content indicators on deformation, and if so, how could these effects best be described. These expectations were based on a relatively large number of experimental settings, in addition to which great care was taken to ensure identical conditions in the method of storing the samples and in the external circumstances.

6. Evaluation of the deformation results

Two different approaches were applied in the evaluation of the test results, as was the case with compressive strength: calculations were made using the concrete composition content indicators (*p*, *x*, χ_c), and again using the traditional, effective *w/c-ratio* (the latter with the *c/R* cement-mass ratio and the χ_c variables).

Furthermore, we evaluated the effect of the *fine particle content* of the aggregates, therefore an additional variable was included in both cases: the f_z/f_a ratio, which is the ratio between the *volumetric specific surface areas of the paste powder* and of the aggregates in the mixture. The *volumetric specific surface areas* of the aggregates were calculated *from the size distribution of particles* by Kausay's method [6], with the difference being that the form factor was given as *5*. For the paste powders, the results of the *Blaine* procedure were used to calculate the volumetric specific surface areas. The *ratios* of the specific surface areas have a practical significance, which we would like to emphasise. In the case of a paste powder of cement and additions with approximately identical levels of fineness, and aggregates where $D_{max} \leq 8$ mm, if $f_z/f_a < 140$, then the aggregate is of *high sand content*, if $140 \leq f_z/f_a < 190$, then it is of *medium sand content*, if $190 \leq f_z/f_a < 450$, then it is of *low sand content*, and if $450 \leq f_z/f_a$, then it is *sand deficient*. The ratio of the *volumetric specific surface areas* can play an important role as a factor not only in shrinkage, but also in the effect on *fresh concrete consistency*.

In view of the fact that measurements were made on the same prisms at different ages - it was given that the age of the concrete (*t* [days]) was also considered as a *variable* - even if only to check the suspicion expressed in connection with *Fig. 16* above.

6.1 Relationship between deformation and concrete composition content indicators

Eq. (35) was found to be suitable for estimating deformation (drying shrinkage) of the prisms made from the experimental mixes (only the *multiplication factor exponential* formula is represented):

$$\varepsilon_{cds}(t) = A \cdot \frac{(1+x)^{n_x}}{\chi_c^{n_\chi}} \cdot \left(\frac{p}{a}\right)^{n_p} \cdot \left(\frac{f_z}{f_a}\right)^{n_f} \cdot (\ln t)^{n_t} \qquad (35)$$

where $\varepsilon_{cds}(t)$ is the expected value, as a percentage, of the drying *deformation* of the prisms, at the age of $28 \leq t \leq 112$ days,

A is the experimental constant,

x is the liquid-powder volumetric ratio in the paste, and n_x is the coefficient of (1+*x*),

χ_c is the cement volumetric ratio in the paste powder, and n_χ is the coefficient of χ_c,

p/a is the volumetric ratio of paste and aggregates in the concrete, and n_p is the coefficient of *p/a*,

f_z/f_a is the ratio of the paste powder and aggregate volumetric surface area, and n_f is the coefficient of *f*, and

t [days] is the age of the concrete in the range $28 \leq t \leq 112$ days, and n_t is the coefficient of ln*t*.

Table 10 summarises the parameters of Eq. (35) for two types of cement, in the ranges given in *Table 9*.

6.2 Relationship between deformation and the *w/c ratio* and *cement content*

If x is replaced by w/c, and p/a is replaced as a variable by the cement ratio c/R, then we reach, from an evaluation of the results, the multiplication factor exponential equation (36):

$$\varepsilon_{cds}(t) = A \cdot \frac{\left(\frac{w}{c}\right)^{n_w} \cdot \left(\frac{c}{R}\right)^{n_c}}{\chi_c^{n_\chi}} \cdot \left(\frac{f_z}{f_a}\right)^{n_f} \cdot (\ln t)^{n_t} \qquad (36)$$

where w/c is the traditional water-cement ratio, and n_w is the coefficient of w/c,

c/R is the cement mass ratio in the concrete (R: concrete density), n_c is the coefficient of c/R,

and the others parameters are the same as in Eq. (35).

Table 10 summarises the parameters of Eq. (36) for two types of cement, in the ranges given in *Table 9*.

6.3 Comparison of the methods for drying shrinkage estimation

The two methods of estimation described above are virtually equivalent in their statistical reliability, as can be seen from the data in *Table 10*.

If we plot the estimated and measured values for deformation at days 28, 56 and 112 for all the prisms, there is hardly any significant difference between the two methods (see *Fig. 18*). From a statistical point of view, there is no visible difference between the two methods.

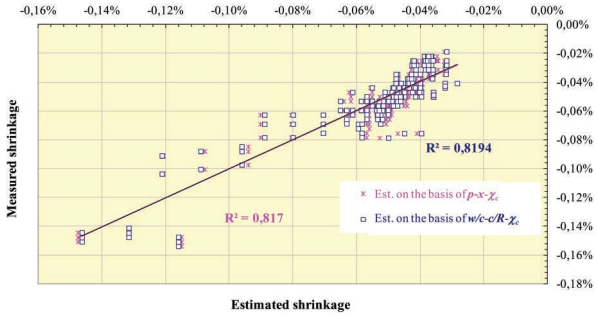

Fig. 18. Deformation of the prisms with CEM III/B 32.5 N-S at days 28, 56 and 112, comparing the values estimated using Eqs. (35) and (36) and the values actually measured, during the ÉMI experiments of 2010.

18. ábra A CEM III/B 32,5 N-S cementből készült próbatestek 28, 56 és 112 napos alakváltozásainak (35) illetve (36) képletekből becsült értékei és a ténylegesen mért értékek között, az ÉMI 2010. évi kísérletei során

7. Discussion of results

If estimation nomograms (*Fig. 19*) are made for the ranges – as of *Table 9* – of the mixes made with CEM III/B 32.5 N-S, where the bands of the same colour between the contours represent (approximately) equal degrees of deformation, it can be seen that the *x-χ_c-p/a-based estimation, in the case of low χ_c values, is progressively less sensitive to an increase in the x liquid-powder volumetric ratio, but more sensitive to a decrease in χ_c, that is, an increase in the content of additions*. The contours of the nomogram in this range are running close together, indicating the instability of the estimation. To estimate deformation, therefore, the estimate based on w/c-c/R-χ_c appears more stable (*Fig. 20*). This creates a strange "reversed analogy" with the estimate for compressive strength, where density among the contours of the nomograms was observed in the w/c-c/R-based estimates, and in that instance the *p-x-χ_c*-based estimate appeared to be the more stable.

The emphasis could be shifted to express the above as follows: increasing the content of additions leads to an increase in shrinkage. This is particularly clear from *Fig. 19*: for extremely low χ_c values (and therefore very high content of additions) the degree of deformation at day 112 is rather considerable, even as much as -0.10%. In the case of mixes made using CEM I 42.5 N, an increase in the content of additions also demonstrably increases the shrinkage.

The effect of sand content is also interestingly revealing. *Figs. 21* and *22* show that if the sand content is significantly increased (compared with the mixes featured in *Figs. 19* and *20*), then if the ratio of f_z/f_a =183 is decreased to f_z/f_a =119, then the shrinkage of mixes with CEM III/B 32.5 N-S increase to a significant extent; for extremely low χ_c values (and therefore high content of additions) the degree of deformation can be as high as -0.15%. Meanwhile, shrinkage of mixes with CEM I 42.5 N – in the ranges investigated – is only minimally influenced by a change in sand content, which is also expressed by the value for the exponential n_f falling close to zero.

Comparing the deformation properties of the two types of cement investigated, it is worth looking at the mixes with practically *no additions* (χ_c=0.99) and *medium sand content* (f_z/f_a =*180-190*) in the range for which measurements could be made for both types of cement (see *Figs. 23* and *24*). It is clear that, in this range, mixes with CEM III/B 32.5 N-S display less shrinkage than those with CEM I 42.5 N.

		Parameters of the Eqs. (35) and (36)						Statistical data	
Estimation on the basis of x_c-x-p /Eq. (35)/	Types of cement	A	n_t	n_x	n_x	n_p	n_f	R^2	Standard Error of the Estimate
	CEM I 42.5 N	-9.8393×10^{-5}	0.8570	0.3432	0.2234	0.4424	0.0700	0.8126	0.0053%
	CEM III/B 32.5 N-S	-7.1735×10^{-3}	0.7172	0.3841	0.2849	0.1056	-0.8021	0.8170	0.0108%
Estimation on the basis of w/c-c/R /Eq. (36)/	Types of cement	A	n_t	n_x	n_w	n_c	n_f	R^2	Standard Error of the Estimate
	CEM I 42.5 N	-3.6727×10^{-4}	0.8579	0.5359	0.3942	0.5425	0.0313	0.7890	0.0057%
	CEM III/B 32.5 N-S	-1.0940×10^{-2}	0.6673	0.2797	0.2881	0.2235	-0.6932	0.8194	0.0107%

Table 10. Parameters of estimation Eqs. (35) and (36) for the two types of cement investigated

10. táblázat A (35) és (36) becslőképletek paraméterei a vizsgált két cementfajtára

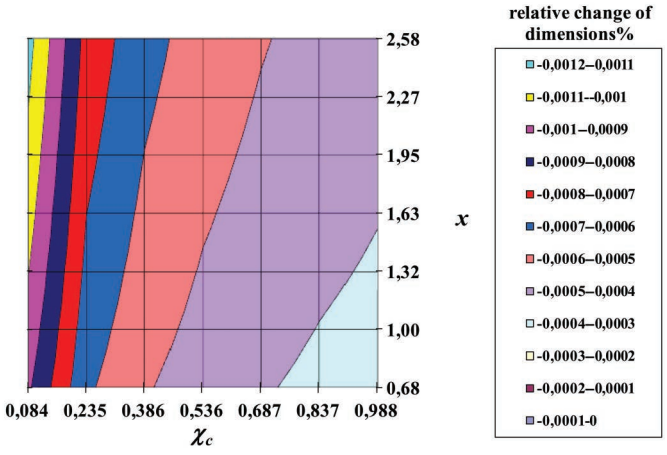

Fig. 19. Nomogram of deformation estimates based on x-χ_c-p/a for mixes made with CEM III/B 32.5 N-S at the age of t=112 days, when p=0.294 and f_z/f_a =183 (medium sand content aggregate)

19. ábra CEM III/B 32,5 N-S cementtel készült keverékek x-χ_c-p/a-alapú alakváltozás-becslő nomogramja t=112 napos korban, p=0,294 és f_z/f_a=183 (közepes homoktartalmú adalékanyag) esetén

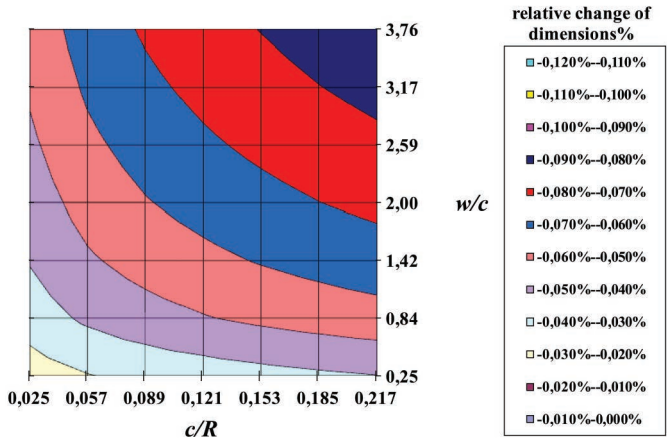

Fig. 20. Nomogram of deformation estimates based on w/c-c/R-χ_c for mixes made with CEM III/B 32.5 N-S at the age of t=112 days, when χ_c=0.99 and f_z/f_a =183 (neat cement paste powder, medium sand content aggregate)

20. ábra CEM III/B 32,5 N-S cementtel készült keverékek w/c-c/R-χ_c-alapú alakváltozás-becslő nomogramja t=112 napos korban, χ_c=0,99 és f_z/f_a=183 (tiszta cementes péppor, közepes homoktartalmú adalékanyag) esetén

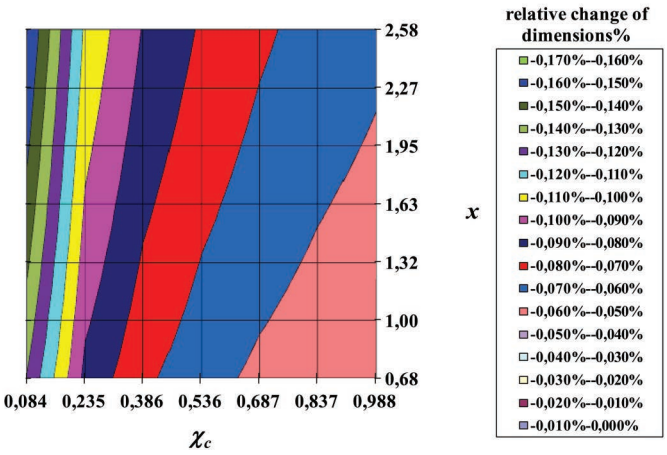

Fig. 21. Nomogram of deformation estimates based on x-χ_c-p/a for mixes with CEM III/B 32.5 N-S at the age of t=112 days, when p=0.294 and f_z/f_a=119 (medium sand content aggregate)

21. ábra CEM III/B 32,5 N-S cementtel készült keverékek x-χ_c-p/a-alapú alakváltozás-becslő nomogramja t=112 napos korban, p=0,294 és f_z/f_a=119 (homokdús adalékanyag) esetén

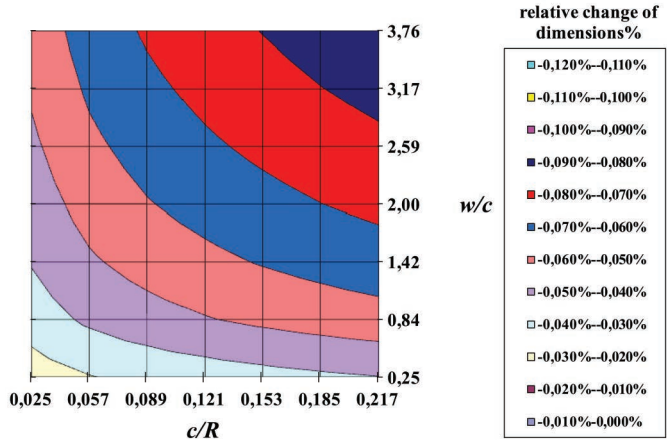

Fig. 22. Nomogram of deformation estimates based on w/c-c/R-χ_c for mixes with CEM III/B 32.5 N-S at the age of t=112 days, when χ_c=0.99 and f_z/f_a=119 (pure cement paste powder, high sand content aggregate)

22. ábra CEM III/B 32,5 N-S cementtel készült keverékek w/c-c/R-χ_c-alapú alakváltozás-becslő nomogramja t=112 napos korban, χ_c=0.99 és f_z/f_a=119 (homokdús adalékanyag) esetén

If, however, 15% v/v (inert stone powder) addition is added to the paste powder, and the aggregate sand content is also increased (f_z/f_a =110-120), then the "tables are turned": while the difference in shrinkage of the two types of cement is slight if the cement content is high, but if the dosage of cement is lower then the mixes with CEM I 42.5 N display demonstrably less shrinkage (see *Figs. 25* and *26*).

If we compare the statements from the literature with our results, there is essentially very close agreement, in addition to which some *novel* facts have come to light regarding the *influence of additions on deformation*, and the *quantification of the effects of the aggregate sand content*.

The effect of concrete composition content indicators on the deformation of hardened concretes can be clearly verified, although it will be necessary to make further measurements in the future to discover, with a degree of certainty that meets the requirements of the modern age, the way in which concrete components of varying properties (cements, additions, aggregates, shrinkage-reducing agents, etc.) and concrete mixes made from them influence deformation.

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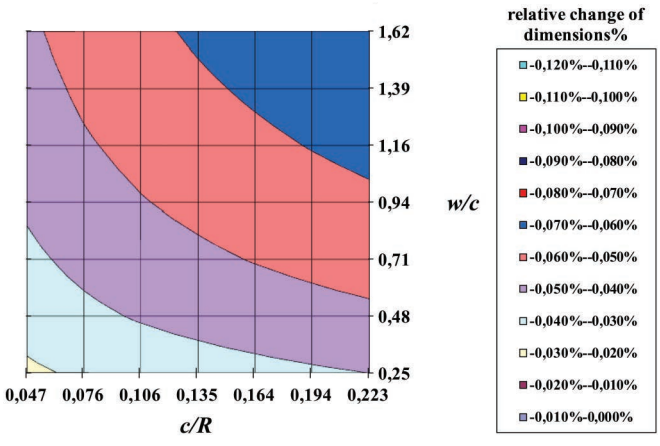

Fig. 23. Nomogram of deformation estimates based on w/c-c/R-χ_c for mixes with CEM III/B 32.5 N-S at the age of t=112 days, when χ_c=0.99 and f_z/f_a =183 (pure cement paste powder, medium sand content aggregate)

23. ábra CEM III/B 32,5 N-S cementtel készült keverékek w/c-c/R-χ_c-alapú alakváltozás-becslő nomogramja t=112 napos korban, χ_c=0,99 és f_z/f_a=183 (tiszta cementes péppor, közepes homoktartalmú adalékanyag) esetén

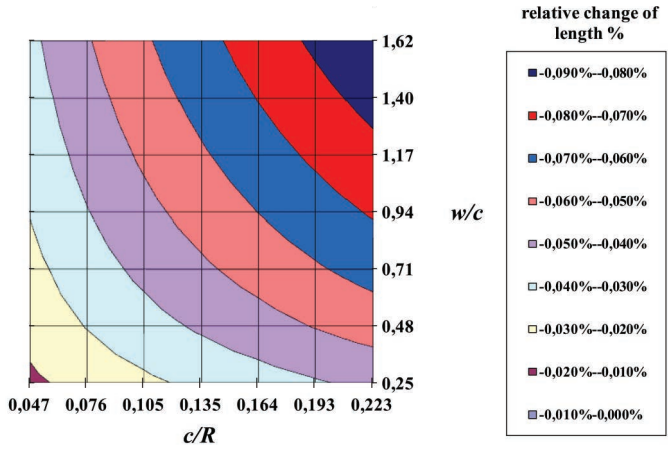

Fig. 24. Nomogram of deformation estimates based on w/c-c/R-χ_c for mixes with CEM I 42.5 N at the age of t=112 days, when χ_c=0.99 and f_z/f_a =186 (pure cement paste powder, medium sand content aggregate)

24. ábra CEM I 42.5 N cementtel készült keverékek w/c-c/R-χ_c-alapú alakváltozás-becslő nomogramja t=112 napos korban, χ_c=0,99 és f_z/f_a=186 (tiszta cementes péppor, közepes homoktartalmú adalékanyag) esetén

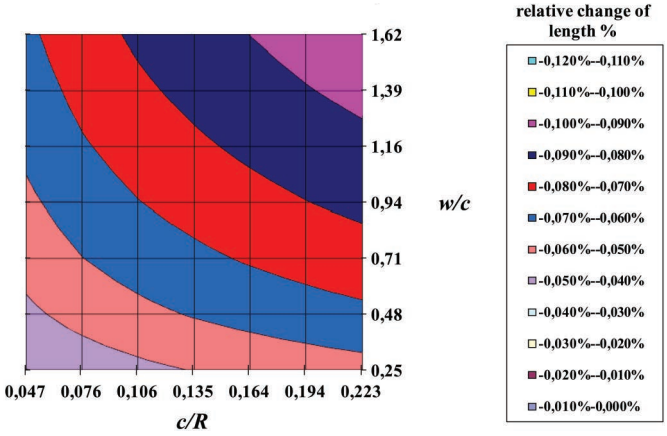

Fig. 25. Nomogram of deformation estimates based on w/c-c/R-χ_c for mixes with CEM III/B 32.5 N-S at the age of t=112 days, when χ_c=0.85 and f_z/f_a =119 (15% v/v additions, high sand content aggregate)

25. ábra CEM III/B 32,5 N-S cementtel készült keverékek w/c-c/R-χ_c-alapú alakváltozás-becslő nomogramja t=112 napos korban, χ_c =0,85 és f_z/f_a =119 (15 tf% kiegészítő, homokdús adalékanyag) esetén

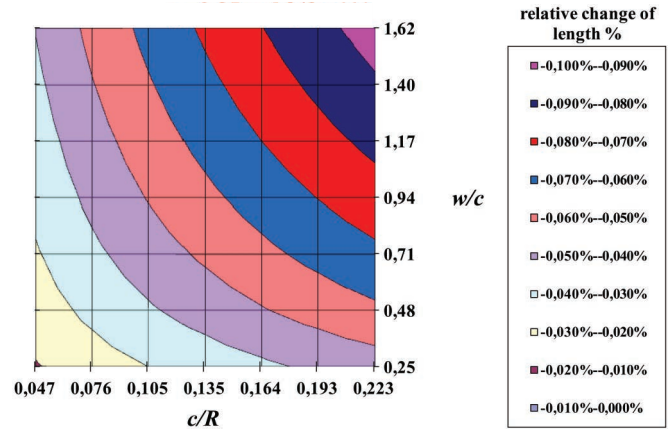

Fig. 26. Nomogram of deformation estimates based on w/c-c/R-χ_c for mixes with CEM I 42.5 N at the age of t=112 days, when χ_c=0.85 and f_z/f_a =111 (15% v/v additions, high sand content aggregate)

26. ábra CEM I 42.5 N cementtel készült keverékek w/c-c/R-χ_c-alapú alakváltozás-becslő nomogramja t=112 napos korban, χ_c =0,85 és f_z/f_a =111 (15 tf% kiegészítő, homokdús adalékanyag) esetén

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Építőanyag, 65. évf. 3. szám (2013), 76–84. p.
http://dx.doi.org/10.14382/epitoanyag-jsbcm.2013.15

Betonkeverékek egyszerűsített alapmodellje és alkalmazása. 2. rész: Nyomószilárdságot és alakváltozást befolyásoló tényezők

A dimenzió nélküli betonösszetételi állapotjelzők bevezetésével a betonkeverékek strukturális összetétele szemléletesen leírható, lehetőséget kínálva a friss és megszilárdult betonkeverékek teljesítményjellemzőit befolyásoló hatások eddigiektől eltérő megközelítésű elemzésére. Cikkünkben megvizsgáltuk a nyomószilárdságot befolyásoló tényezőket és az elemzés révén közelebb jutottunk a szakirodalomban említett két, eddig nem értelmezett probléma magyarázatához. A laboratóriumi kísérletek során készült kisméretű próbahasábok alakváltozásainak vizsgálati eredményeit is ismertettük, és az elemzés során összefüggéseket keresünk a betonösszetételi állapotjelzők és a megszilárdult betonok alakváltozása között. Érdekes párhuzam mutatkozik a szilárdságot és az alakváltozást befolyásoló hatásokban, és lehetőség nyílik arra, hogy az alakváltozás is előre becsülhető a betonösszetételi állapotjelzőkből.

Kulcsszavak: betontechnológia, betonösszetétel tervezése, betonösszetételi állapotjelzők

CSHub MIT CONCRETE SUSTAINABILITY HUB

Why Concrete

Simple. More concrete is produced than any other material on Earth. In the foreseeable future, there is no other material that can replace concrete to meet our societies' needs for housing, shelter, schools, and infrastructure. Concrete is an essential part of **LIFE**.

The MIT Concrete Sustainability Hub, CSHub, is a dedicated team of interdisciplinary researchers from several MIT departments working on concrete and infrastructure science, engineering, and economics since 2009. The MIT CSHub brings together leaders from academia, industry, and government to develop breakthroughs using a holistic approach that will achieve durable and sustainable homes, buildings, and infrastructure in ever more demanding environments.

The CSHub is focused on Building for **LIFE: Life cycle thinking * Innovation * Fiscal responsibility * Environmental leadership**.

Concrete is produced from abundant raw materials locally available almost everywhere on earth. It is an inexpensive construction material with a relatively small environmental footprint, but its attractive properties have lead to massive use that contributes approximately 5% of global CO_2 production.

Emerging breakthroughs in concrete science and engineering hold the promise that concrete can be part of the solution of contributing to a sustainable development that encompasses economic growth, social progress while minimizing the ecological footprint. This requires a holistic approach in which progress in concrete science seamlessly feeds into innovative structural concrete engineering applications, ranging from concrete pavement solutions to wall systems, whose impact on sustainable development are evaluated with advanced environmental-economic impact studies.

The mission of the CSHub is to develop breakthroughs that will achieve sustainable and durable homes, buildings, and infrastructure. The approach is to conduct research to reduce the impact of producing and using concrete, and to develop tools to support infrastructure decisions: life-cycle environmental, cost, and hazard resistance.

http://cshub.mit.edu/

Lead (II) and zinc (II) ions removal capacity of coarse limestone and rhyolite tuff from aqueous solutions

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Abstract
The heavy metal adsorption capacity of natural stones is important because it has influence on the dispersion of pollutants. Lead (II) and zinc (II) ions removal capacity of two Hungarian stones (coarse limestone and rhyolite tuff) were analysed under laboratory conditions. Petrophysical parameters of rock samples were determined at air-dried and in water saturated conditions: apparent density, capillary water absorption, ultrasound pulse velocity, open and full porosity. The powdered rock samples and specimens were put into lead-nitrate and zinc-sulphate solutions and the amounts of adsorbed lead (II) and zinc (II) ions were identified by titration of the residual solution. According to the tests, the powdered and cylindrical rock specimens could reduce the concentration of lead (II) and zinc (II) ions in the heavy metal solutions. The results suggest that these two types of rocks could be used in environmental protection technologies such as material of permeable reactive barrier.
Keywords: lead (II), zinc (II), coarse limestone, rhyolite tuff, environmental protection

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1. Introduction

Waste water with heavy metal content is considered to be hazardous both for human life and for the environment due to their acute toxicity and non-biodegradability. In the last few decades several technologies have been developed to remove heavy metal ions from industrial waste water and prevent the environment from heavy metal pollution. Despite of the high efficiency of the applied technologies (like using activated carbon), these are very expensive and in some cases continuously chemical input and/or pre-treatment is needed. The removals of metals are not always incomplete and the follow-up treatment of the used adsorber materials is not highly elaborated [1]. Therefore, the search of low-cost alternative adsorbents is essential.

Some materials, like porous natural stones, could immobilize heavy metals from industrial waste water and from polluted groundwater according to their chemical and physical properties. Chemical features (especially mineralogical composition) of limestone and rhyolite tuff could bind certain contaminants (with adsorption or chemical precipitation on the mineral surface), and subtract them from the natural geochemical cycle. Physical characteristics (porosity, permeability, and pH) of these two stones could influence the accessibility of the solutions which contain heavy metals [2–5]. These properties of natural porous rocks may prefer to use in many industrial areas, such as waste water and flue gas cleaning processes, as well as agricultural practices of soil improvement. Recently, natural stones are used as an adsorbents in permeable reactive barriers in polluted groundwater cleaning processes [6–9] and also as host rocks of hazardous waste deposits [10, 11].

Lead and zinc are one of the most common contaminants of industrial waste waters. Storage batteries, printing, painting dying, mining, metallurgy and fuel combustion are the source of these heavy metals [12].

Lead and zinc poisoning could cause health problem, such as the damage of liver and kidney, mental retardation, neurological problems. Therefore the reduction of the concentration of lead, zinc and other heavy metal ions in waste water is very indispensable.

The aims of this paper were to observe the spread of heavy metal solution in Hungarian porous coarse limestone and rhyolite tuff, determine the effect of these two rocks on solutions with lead and zinc content, and recommend environmental protecting applications of the examined porous rocks.

2. Materials and methods

2.1 Materials

In this study natural rhyolite tuff and coarse limestone were used as adsorbent materials (*Fig. 1*).

The rhyolite tuff of Demjén, from Bükkalja (southwest of Eger), is a grey, pumice (centimeter-sized) rich and Miocene pyroclastic rock. In matrix biotite and quartz could be recognized. Three typical tuff scattering levels (lower, middle and upper) could be found at Bükkalja. The examined rhyolite tuff was from the middle tuff level. The thickness range of this tuff level is between 2-20 m [13].

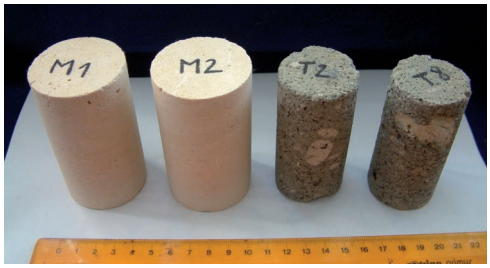

Fig. 1. Cylindrical coarse limestone and rhyolite tuff specimens
1. ábra Hengeres durva mészkő és riolittufa próbatestek

The coarse limestone of Sóskút is a Miocene sedimentary rock. According to its mineralogy and texture, several variations could be identified. During our examination we used the yellowish-white, fine grained ooid rich version. The typical texture of Sóskút coarse limestone is ooid grainstone/packstone [14] by microscopic studies. This limestone could be found in two formations: the most common Tinnyei Limestone Formation (Sarmation) and Rákosi Limestone Formation (Baden). The largest volume of this limestone was mined at Sóskút, Biatorbágy and nearby the capital city of Hungary. It used like building materials of historical monuments in Budapest [15].

2.2 Petrophysical examination of rock samples

We determined basic petrophysical parameters of rock samples in water saturated and air-dried condition: apparent density, capillary water absorption, ultrasound pulse velocity, open and full porosity.

2.3 Preparation of rock samples

The preparation of rock samples was made in two steps. First of all, the coarse limestone and rhyolite tuff were grounded in an agate mortal. The grain size was less than 100 µm.

The examinations were carried out by 5 g powdered rock samples (coarse limestone, rhyolite tuff) in contact with 25 ml of pH neutral heavy metal solutions. The solutions were prepared from basic salts: lead-nitrate $Pb(NO_3)_2$ and zinc-sulphate $ZnSO_4$. The concentrations of each solution were 1000 ppm and the temperature was 22 °C. After 24 hours, powdered samples were filtered through semi-coarse filter paper and dried at 50 °C. The residual heavy metal solutions were collected in plastic bottles until analytical examinations.

In the next step the cylindrical rock specimens (height: 8 cm, diameter: 4 cm) were drilled by diamond drill-bit from coarse limestone and rhyolite tuff. Dried rock specimens were soaked into lead-nitrate or zinc-sulphate heavy metal solutions. The concentrations of the solutions were 1000 ppm, and the temperature was 22 °C. After the 24 hours retention time, the rock specimens were dried at 50 °C and the residual heavy metal solutions were stored also in plastic bottles.

2.4 Wet chemical analysis

After the preparation of the rock samples we determined the concentration of the solutions with heavy metal content by titrimetric (chelatometry) analytical method. During the examination the concentrations of the Complexon solutions were 0.005 and 0. 01 M.

2.5 Characterization of raw and treated materials

Characterization of untreated and treated coarse limestone and rhyolite tuff was carried out by X-ray diffraction (XRD). We used Philips diffractometer (generator: PW 1130; goniometer: PW1050; npd Control counter: PW3710, copper anode and monochromator). Accelerating voltage: 40kV, amperage: 30 mA, measuring angle: 3-70°. The evaluation was made by Philips PW 1877 Automated Powder Diffraction (Version 3.5B) software.

3. Results

3.1 Petrophysical properties of examined porous rocks

According to the petrophysical studies the coarse limestone's apparent density, ultrasound pulse velocity and capillary water absorption were larger than the rhyolite tuff's, but the full porosity was less. According to the results, the cause of the difference is the larger amount of pumice in rhyolite tuff. The pore system of pumice is closed, so the water could not migrate so deeply in the stone. This is the reason, why the capillary water absorption and the full porosity values are lower in rhyolite tuff than in coarse limestone. Pores of pumice are mainly filled by air and have bigger pore diameter, so the ultrasound pulse velocity of rhyolite tuff is reduced. Because of the low density of pumice (< 1 g/cm³) the high amounts of it, could decrease the apparent density of rhyolite tuff. Nevertheless, the two rock open porosity values were high (> 30%) and approximately the same. This result suggests, that both types of rock are sufficiently porous to use as migration passage of the experiment solutions (*Table 1*).

Tests	Coarse limestone	Rhyolite tuff
Apparent density (g/cm³)	2.70	2.45
Ultrasound pulse velocity (m/s)	2580-3165	1921-2255
Capillary water absorption (kg/m²)	27-26.9	21.1-21.4
Open porosity (%)	30.1	30.2
Total porosity (%)	30.2	39.4

Table 1. Results of petrophysical analysis
1. táblázat Kőzetfizikai vizsgálatok eredményei

3.2 Wet chemical analysis

In accordance with wet chemicals analysis 1 kilogram of powdered rock samples could immobilize 5-10 times more lead (II) and zinc (II) ions from heavy metal solutions than 1 kilogram of cylindrical rock specimens (*Fig. 2* to *3*). These phenomena could be explained by the greater specific surface area of powdered rock samples than cylindrical rock specimens.

It was also found, that powdered rock samples could bind the same quantity of heavy metals (lead (II) and zinc (II)) against to rock specimens. The cause is the mineralogical content of the examined rocks. Rhyolite tuff contains a lot of glass beside other minerals, like biotite, quartz, feldspars. Glass has several reactive ligands, which can bind heavy metals, like lead and zinc also. This is the same situation with pumice. Demjén rhyolite

tuff includes many pumice fragments, which are built up mostly from glass. This kind of clasts could increase the binding features of rhyolite tuff. In contrast, coarse limestone is built up only from calcite minerals. This rock is quite homogeneous. Calcite has less reactive functional groups, which could bind heavy metals, but it does not mean that this mineral is not able to do it.

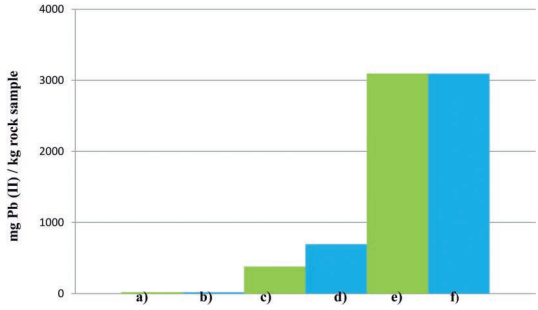

Fig. 2. Lead (II) immobilization from lead-nitrate aqueous solution (1000 ppm) by rock samples in various physical conditions: a) untreated coarse limestone, b) untreated rhyolite tuff, c) soaked coarse limestone specimens, d) soaked rhyolite tuff specimens, e) powdered and soaked coarse limestone, f) powdered and soaked rhyolite tuff

2. ábra Különböző fizikai állapotú kőzetminták ólom (II) megkötő képessége ólom-nitrát oldatból (1000 ppm): a) oldattal nem kezelt durva mészkő; b) oldattal nem kezelt riolittufa, c) oldatba áztatott durva mészkő próbatestek, d) oldatba áztatott riolittufa próbatestek, e) porított és oldatba áztatott durva mészkő, f) porított és oldatba áztatott riolittufa

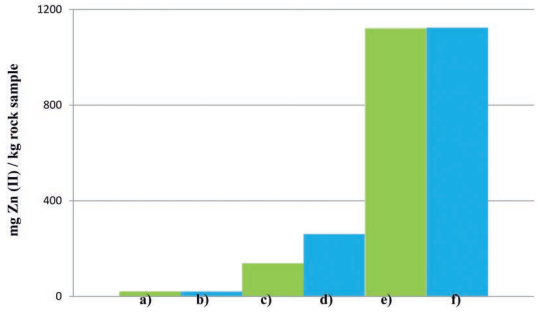

Fig. 3. Zinc (II) immobilization from zinc-sulphate aqueous solution (1000 ppm) by rock samples in various physical conditions: a) untreated coarse limestone, b) untreated rhyolite tuff, c) soaked coarse limestone specimens, d) soaked rhyolite tuff specimens, e) powdered and soaked coarse limestone, f) powdered and soaked rhyolite tuff

3. ábra Különböző fizikai állapotú kőzetminták cink (II) megkötő képessége cink-szulfát oldatból (1000 ppm): a) oldattal nem kezelt durva mészkő; b) oldattal nem kezelt riolittufa, c) oldatba áztatott durva mészkő próbatestek, d) oldatba áztatott riolittufa próbatestek, e) porított és oldatba áztatott durva mészkő, f) porított és oldatba áztatott riolittufa

According to wet chemical analysis, rhyolite tuff specimens could bind much bigger quantity of heavy metals (lead (II) and zinc (II)), than coarse limestone specimens (*Fig. 2* to *3*). It has a physical explanation. Coarse limestone has much smaller pore diameter, than rhyolite tuff. That's why pores in coarse limestone could be clogged by precipitated minerals (nitrate and sulphate salts) and the migration of heavy metal solution could stop. In this case high amount of the minerals in coarse limestone do not have a chance to contact with heavy metals, and the immobilization will not happen.

It is also important, that both of the examined rocks (in powdered or in specimens form) could bind more lead (II) than zinc (II) ions (*Fig. 2* to *3*). It could be explained by the difference of mobility between zinc (II) and lead (II) ions. In aqueous solution zinc (II) ions are more mobile than lead (II) ions. The rate of immobilisation on the solid phase (minerals), in the case of lead (II) ions, is higher than zinc (II) ions.

3.3 Characterization of raw and treated materials

The X-ray diffraction examinations suggest that untreated rock samples did not contains any lead or zinc mineral phases (*Fig. 4* to *5*). After wet chemical preparation it was found that coarse limestone did not contain heavy metal ions (*Fig. 5*), but rhyolite tuff contained lead and zinc mineral phases (*Fig. 4*). It suggests, that heavy metal removal was taken place by chemical precipitation (lead or zinc mineral phases) in rhyolite tuff. Despite the facts, that lead and zinc mineral phases were not determined by X-ray diffraction in coarse limestone, it does not mean that coarse limestone was not able to immobilize heavy metals from their solution. Wet analysis proved that coarse limestone could bind heavy metals from their aqueous solutions, but probably the process did not take place by chemical precipitation, but adsorption. This type of physical adsorption could not demonstrated by X-ray diffraction. This method is usable for phase analysis. This theory needs to be proved by scanning electron microscope (SEM) observations in the future.

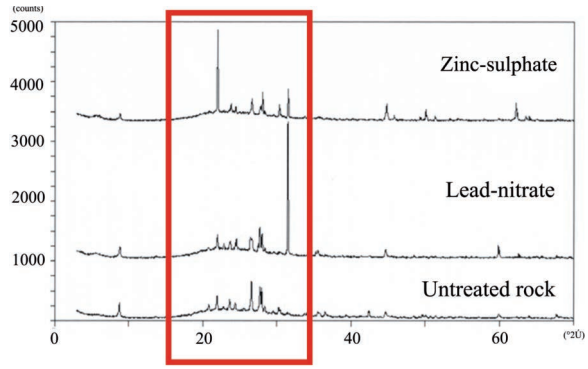

Fig. 4. X-ray diffractogram of rhyolite tuff

4. ábra Riolittufa röntgen diffraktogramja

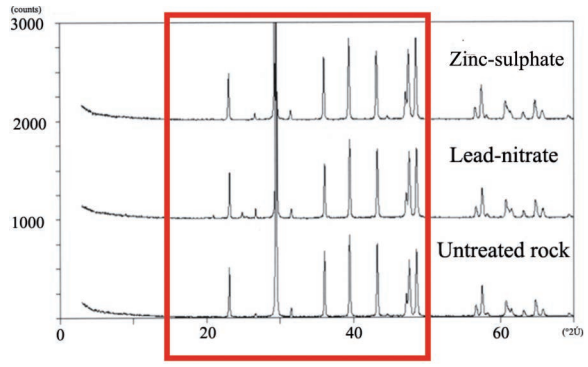

Fig. 5. X-ray diffractogram of coarse limestone

5. ábra Durva mészkő röntgen diffraktogramja

4. Conclusions

Lead (II) and zinc (II) ions could be very successfully removed from aqueous solutions by chemical precipitation and adsorption process on Demjén rhyolite tuff and Sóskút coarse limestone. Lead (II) and zinc (II) ions were bound from aqueous solution by both examined rocks, but heavy metal removal capacity of rhyolite tuff specimens was greater.

Lead (II) ion removal efficiency was higher than zinc (II) ion immobilization in both types of rocks. The binding capacity of powdered rock sample was greater than rock specimens. According to X-ray diffraction analysis, heavy metals were mainly immobilized by chemical precipitation (in crystal form), but physical adsorption processes are not excluded.

Rhyolite tuff from Demjén and coarse limestone from Sóskút are widespread in Hungary. They are readily available, relatively affordable raw materials and according to our investigations these natural stones have quite high heavy metal removal efficiency from aqueous solution, with high concentration. Depending on the desired reduction of heavy metal concentration in aqueous systems, we recommended these two rocks as adsorbent materials in applied environmental processes, such as treating of polluted ground water by permeable reactive barrier. Instead of the raw coarse limestone and rhyolite tuff, we also suggest the application of used building stones and materials too.

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Ref.:
Gabriella **Németh** – Lilla **Mlinárik** – Ákos **Török**: *Lead (II) and zinc (II) ions removal capacity of coarse limestone and rhyolite tuff from aqueous solutions*
Építőanyag, 65. évf. 3. szám (2013), 86–89. p.
http://dx.doi.org/10.14382/epitoanyag-jsbcm.2013.16

Riolittufa és durva mészkő oldott állapotú ólom (II) és cink (II) ion megkötő képességének vizsgálata

A kőzetek nehézfém megkötő képessége igen fontos az előforduló környezeti szennyeződések terjedésének befolyásolásában. Vizsgálataink során két hazai kőzettípus (durva mészkő és riolittufa) ólom (II) és cink (II) megkötő képességét vizsgáltuk laboratóriumi körülmények között. A vizsgálatok során meghatároztuk a kőzetek kőzetfizikai paramétereit víztelített és légszáraz állapotban: anyagsűrűség, kapilláris vízfelvétel, ultrahang terjedési sebesség, nyílt és összporozitás. A porított, valamint a kőzetekből fúrt próbatesteket ólom-nitrát, illetve cink-szulfát oldatba áztattunk, majd nedves analitikai (kelatometriás) módszerrel meghatároztuk a visszamaradt oldat ólom (II) és cink (II) ion koncentrációját. A vizsgálatok szerint a porított kőzetminták, és a kőzetekből fúrt hengeres próbatestek is képesek voltak az ólom (II) és cink (II) ionok koncentrációját csökkenteni a nehézfém tartalmú oldatokban. Az eredmények alapján megállapítható, hogy a vizsgált két kőzet alkalmas lehet későbbi környezetvédelmi eljárásokban való alkalmazásra, többek között, mint reaktív gátanyag.

Kulcsszavak: ólom (II), cink (II), durva mészkő, riolittufa, környezetvédelem

Performance of waste glass powder (WGP) supplementary cementitious material (SCM) – Workability and compressive strength

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Abstract
Ecological and environmental benefits support the use of waste glass powder (WGP) as supplementary cementing material by the decrease of the amount of landfills, by the reduction of non-renewable natural resource consumption, by the reduction of energy demand for cement production (less cement is needed), and the reduction of greenhouse gas emission. Laboratory tests were carried out on cement paste specimens, in which waste glass powder (WGP) addition was used as a supplementary cementitious material. Cement was substituted with WPG at levels of 20% or 30% per mass. It was demonstrated that the WGP addition improves the workability of fresh pastes, and can be effectively used as cement replacement for compressive strength. It was also demonstrated that the particle size of the WGPs (specific surface area) has a stronger influence on the effectiveness of the cement replacement than the chemical composition. The effectiveness of the cement replacement increases as the specific surface area increases.
Keywords: recycling, waste glass, supplementary cementitious material, workability, compressive strength

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1. Introduction

It is a challenge in civil engineering to transform the industrial wastes into construction material components. The increasing amount of unmanaged wastes has resulted in a critical environmental impact. The recycling of industrial wastes and the use of them as construction materials can provide a promising solution from both economy and ecology point of view, by the decrease of pollution and by the more economic design and architecture. Environmentally friendly and low-cost construction materials receive increasing attention recently, due to their attractiveness as building materials benefiting to the environment and promoting sustainability in the building construction industry.

Concrete is the most commonly used construction material in the planet and it is the second most consumed product after water [1].

As of 2012, the annual global cement production is over 3.6 billion tons, and is expected to be increased soon to over 4 billion tons per year, nevertheless, the cement industry is confronting with the continuous increase in cost for energy supplies, the obligations to reduce CO_2 emission and the need of appropriate supply of raw materials both in quality and quantity [2,3]. It is estimated that about 0.9–1.0 tons of CO_2 are produced for a ton of clinker depending on the type of fuels used [4].

Nowadays, the clinker content of cements is intensively replaced by supplementary cementitious materials. Fly ash, blast furnace slag, natural pozzolans and limestone are used in increasing amounts that can substitute some clinker in the cement. *Fig. 1* illustrates how the proportions of OPC declined over time in relation to the cements with other major constituents [2]. As a result, an average specific CO_2 emission per ton of cement today is 20% lower compared to the 1990s.

Ecological and environmental benefits of the supplementary cementitious materials are 1) non-recycled waste is used rather than increase the amount of landfills, 2) the consumption of non-renewable natural resources is reduced, 3) the energy needed for cement production is reduced, 4) the emission of greenhouse gases is reduced [5].

2. Waste glass for supplementary cementitious material

Soda-lime glass is the most widely used glass for both packaging and in the building construction, however, based on chemical composition, glass can be categorized into several other categories as well. Containers, float and sheet glass are mostly soda-lime glass and, as a consequence, industrial waste glass is composed over 80% by weight of soda-lime glass [6]. For soda-lime glass the typical composition is approximately: 70% SiO_2, 13–17% Na_2O and 10% CaO.

The characteristics of soda-lime glass make it suitable for consideration as an aggregate to concrete or as a supplementary cementitious material (SCM), but the high alkali content of soda-lime glass is one typical concern for the use in concrete. The possible formation of the harmful alkali-silica reaction (ASR) in contact with Portland cement may limit the

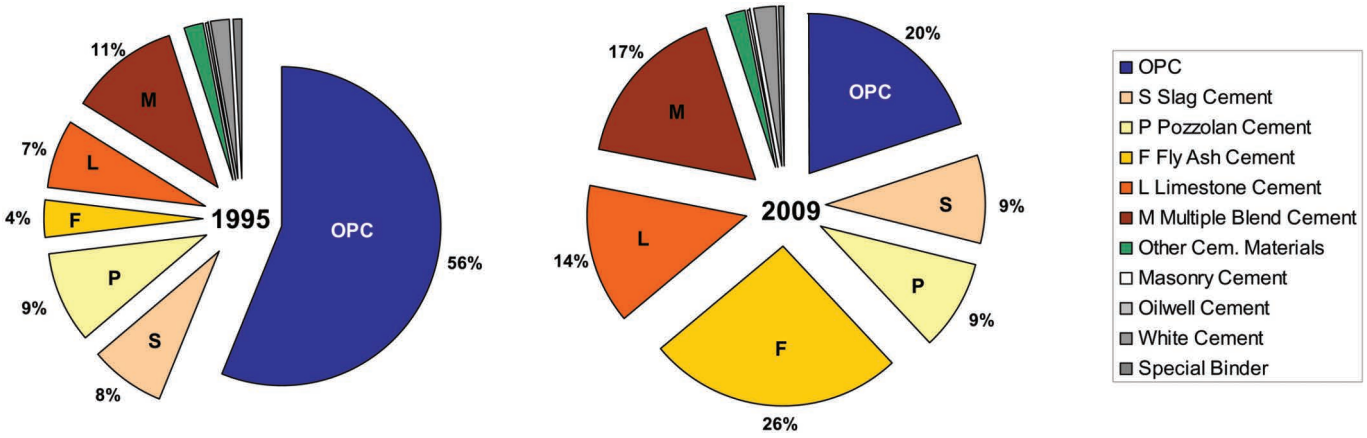

Fig. 1. Proportions of supplementary materials in cements [2]
1. ábra Cement kiegészítő anyagok alkalmazott mennyiségének változása [2]

applications. Alkalis can cause alkali-aggregate reaction and expansion if the aggregate of the concrete is reactive.

Experimental results have demonstrated that both particle size and glass colour have an influence on the expansion during alkali-silica reaction [7]. Generally, the finer glass particles exhibit considerably lower expansion; pozzolanic activity increases as fineness increases. Fine particles of waste glass powder (WGP) also tend to perform a relatively rapid pozzolanic reaction with Portland cement on the contrary to the much slower alkali-silica reaction. The combined use of other supplementary cementing materials such as fly ash, ground blast furnace slag and metakaolin can also decrease the expansion from ASR.

The pozzolanic properties of waste glass powder (WGP) are first notable at particle sizes below approximately 300 μm. Below 100 μm, glass can have a pozzolanic reactivity which is greater than that of fly ash. When ground to about the same fineness as Portland cement, waste glass powder (WGP) has advantageous pozzolanic behaviour: the amorphous silica (SiO_2) reacts with portlandite ($Ca(OH)_2$) generated during cement hydration to form gel of calcium silicate hydrate (CSH). No alkali-silica reaction was detected with particle size below 100 μm [6].

It was found experimentally that 30% waste glass powder (WGP) could be incorporated as cement replacement in concrete without any long term disadvantageous effect [8].

Several studies have shown further beneficial effects, including increased workability and reduced permeability, of using WGP as a supplementary cementitious material (SCM), but the compressive strength is often lower when WGP is used as cement replacement, especially at early ages. Cement replacement with waste glass powder was shown to reduce early-age compressive strength, concrete mixtures with glass powder reached strength values close to the strength values for control mixtures at 28 days, and only one glass powder mixture reached a greater strength than the corresponding control at 91 days according to [9].

It was also shown that cement mortar with 20% waste glass powder replacement gained significant strength between 28 and 90 days showing pozzolanic reaction taking place in this period [10]. The continued strength development clearly indicates the beneficial pozzolanic reaction of the glass powder [11].

A reduction in the 28 days compressive strength of about 15% was observed when 20% of cement was replaced by waste glass powder [10,12].

3. Experimental studies

Laboratory tests were carried out on cement paste specimens, in which waste glass powder (WGP) addition was used as a supplementary cementitious material (SCM) during a cooperation research between the Budapest University of Technology and Economics (BME), Department of Construction Materials and Engineering Geology and the Riga Technical University (RTU), Institute of Materials and Structures (IMS), Department of Building Materials and Products..

3.1 Materials

For the specimens, CEM I 42.5 N Portland cement was used provided by a Hungarian cement manufacturer, with a specific surface area of 344 m^2/kg. The WGP addition materials were prepared in RTU IMS laboratory directly for the present experiments, using waste glass cullet collected in Latvia. Five different WGPs were studied. Fluorescent lamp tube glass waste cullet (LB) and incandescent light bulb borosilicate glass waste cullet (DRL) were received from a lamp recycling centre in Liepaja, Latvia. Container glass was obtained as bottles in green (G), amber (A) and flint (F) colours which were collected at a glass bottle return point in Riga, Latvia, and were manually crushed into cullet under laboratory conditions. The cullet was washed, dried and ground for 30 minutes in a laboratory planetary ball mill (Retsch PM400) with rotation speed 300 min^{-1}. The specific surface area of the WGP was obtained by a Zwick/Roell ToniPERM automatic Blaine apparatus; further details of the WGP preparation are available in [13].

Earlier experiences have indicated that the five WGPs used in the present studies can serve as supplementary cementitious materials. It was demonstrated on concrete specimens [14] that WGP addition improves workability and results a softer consistency (demonstrated by standard slump tests), can contribute to the compressive strength of the hardened concrete and due to the relatively high specific surface area (maximum particle size was found to be smaller than 200 μm in each case [13]) they have a relatively small influence on ASR expansion.

Fig. 2. Consistency tests according to EN 1015-3:1999 [16]
2. ábra Konzisztencia vizsgálata az EN 1015-3:1999 szerint [16]

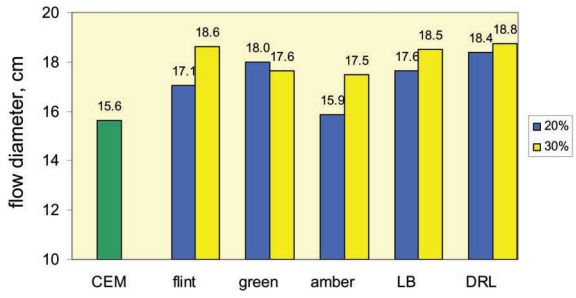

Fig. 3. Consistency tests results
3. ábra Konzisztencia vizsgálatok eredményei

Fig. 4. Compressive strength tests according to EN 196-1:2005 [15]
4. ábra Nyomószilárdság vizsgálata az EN 196-1:2005 szerint [15]

The target of the present series of experiments was the direct analysis of the net influences of the WGPs on the hydration process of Portland cement, therefore, neat cement pastes were prepared with WGP addition. Cement was substituted with WPG at levels of 20% or 30% per mass of cement.

3.2 Mixing and consistency

The control cement paste mixture and the different paste mixtures with WGP addition were mixed in a laboratory mortar mixer according to EN 196-1:2005 [15]. The water/cement ratio was selected to be w/c = 0.285. Where cement replacement by WGP was applied, the water/binder ratio was changed to w/b = 0.342 (20% WGP) and w/b = 0.3705 (30% WGP), however, the water/cement ratio was kept constant at w/c=0.285.

The consistency was tested by a standard flow table (*Fig. 2*) according to EN 1015-3:1999 [16]. The flow table tests were repeated twice for each mixture. After the 15 drops of the flow table (one drop in 2 seconds) the final diameter of the sample was measured in two directions by a steel ruler. Flow table test results are indicated in *Fig. 3*. It can be seen that the WGP addition increases the flow of the fresh pastes. The higher is the amount of the WGP, the higher is the improvement in workability.

3.3 Compressive strength

For the compressive strength tests, 30 mm size cubic specimens were prepared in steel moulds. During the first 14 days of curing period specimens were kept in water in the climatic chamber, after 14th days half of specimens was stored in the climatic chamber (at temperature 20±2 °C and relative humidity >65%) and half of the specimens was kept continuously under water. Compressive strength of cement paste specimens was determined at the age 2, 7, 14, 28, 91, 147 and 302 days (Fig. 4) according to EN 196-1:2005 [15]. The evolution of the compressive strength of the specimens stored under water is indicated in *Fig. 5.* where the hardened neat cement paste is indicated with red colour.

4. Discussion of strength results

The development of the compressive strength is indicated in separate diagrams for the five GWPs in comparison to the neat cement paste (see *Fig. 6.a, 7.a, 8.a, 9.a, 10.a*). Strength development of the hardened neat cement paste at the age of *t* days, is indicated with red colour in each diagram. It can be generally concluded that the cement replacement by GWP was successful form a compressive strength point of view and no significant reduction of the compressive strength can be realised, especially for the mature specimens.

The effectiveness of the cement replacement can be characterised more precisely if the relative development of the compressive strength is studied. *Fig. 6.b, 7.b, 8.b, 9.b, 10.b* indicate the relative compressive strength values corresponding to the different WGPs. The level of 1.0 indicates the compressive strength of the hardened neat cement paste at the age of *t* days, as a reference level in each diagram (indicated with red colour). It can be seen that the compressive strength of the pastes containing WGP addition reaches that of the reference hardened neat cement paste at the age of 28 to 91 days, according to the present laboratory tests. The significance of the WGP amount is also visible: usually, the 20% replacement resulted higher compressive strength values than the 30% replacement. It seems that only a part of the WGP can be activated by ordinary Portland cement during the period of the present experiments (~ one year). A rapid development of the compressive strength is visible between the ages of 28 to 91 days, being faster than that of the hardened neat cement paste that is attributed to a supposed very active hydration of the WGPs during that period of time. At later ages the relative rate of hydration is balanced between the cement and the WGPs, resulted in a decelerated rate and a constant difference in compressive strength values.

It can be also demonstrated that the particle size of the WGPs has a stronger influence on the effectiveness of the cement replacement than e.g. the chemical composition. *Fig. 11* indicates

the relative compressive strength values corresponding to the age of 302 days over the specific surface area of the WGPs used in the present experiments, according to [13]. The effectiveness of the cement replacement increases as the specific surface area increases. The flint WGP seems to be an outlier during the present tests. The results further confirm the importance of the preparation of WGPs with high specific surface area for the optimal supplementary cementitious material performance.

5. Conclusions

The present paper has summarised the experimental results of a laboratory test series carried out on cement paste specimens, in which waste glass powder (WGP) addition was used as a supplementary cementitious material (SCM) during a cooperation research between the Budapest University of Technology and Economics (BME), Department of Construction Materials and Engineering Geology and the Riga Technical University (RTU), Institute of Materials and Structures (IMS). CEM I 42.5 N Portland cement was used with WPG substitution at levels of 20% or 30% per mass of cement.

It was demonstrated that the WGP addition:

- improves the workability of fresh pastes,
- can be effectively used as cement replacement for compressive strength.

It was demonstrated that the particle size of the WGPs (specific surface area) has a stronger influence on the effectiveness of the cement replacement than the chemical composition.

6. Acknowledgement

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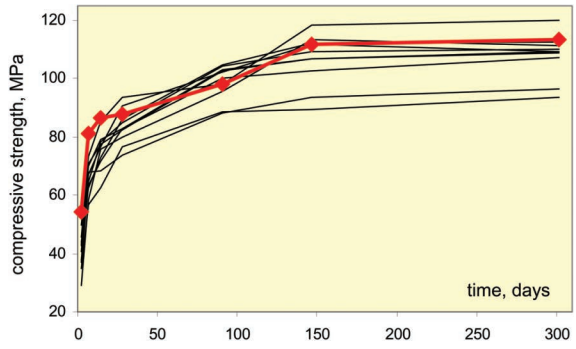

Fig. 5. Evolution of the compressive strength of the specimens stored under water
5. ábra Nyomószilárdság időbeli fejlődése a végig víz alatt tárolt mintákon

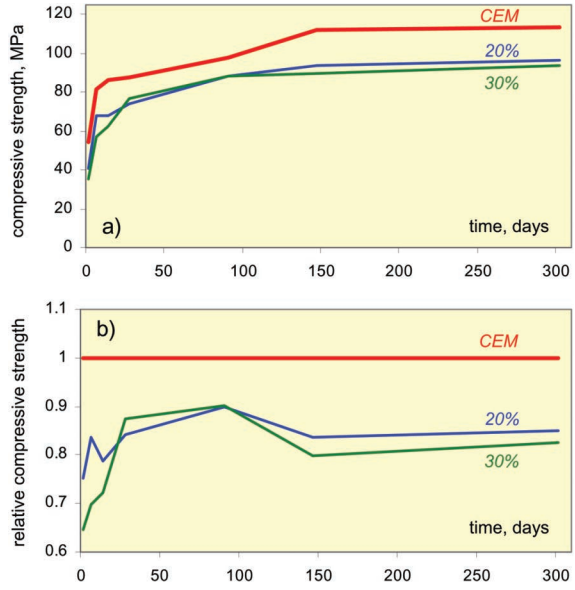

Fig. 6. Compressive strength and relative compressive strength of specimens with flint WGP
6. ábra Nyomószilárdság és relatív nyomószilárdság fejlődése a fehér színű hulladék üveg por kiegészítővel

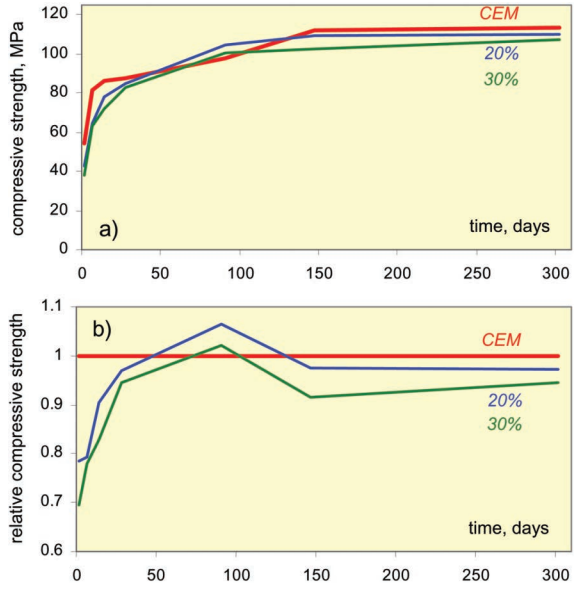

Fig. 7. Compressive strength and relative compressive strength of specimens with green WGP
7. ábra Nyomószilárdság és relatív nyomószilárdság fejlődése a zöld színű hulladék üveg por kiegészítővel

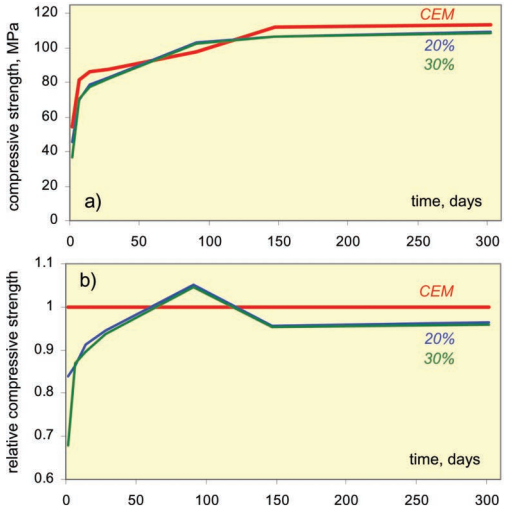

Fig. 8. Compressive strength and relative compressive strength of specimens with amber WGP
8. ábra Nyomószilárdság és relatív nyomószilárdság fejlődése a borostyán színű hulladék üveg por kiegészítővel

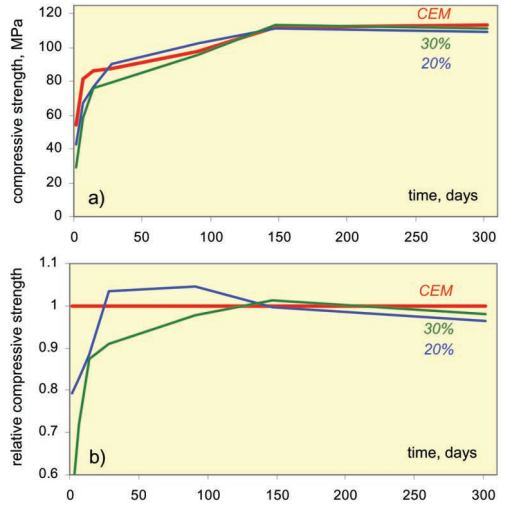

Fig. 9. Compressive strength and relative compressive strength of specimens with LB WGP
9. ábra Nyomószilárdság és relatív nyomószilárdság fejlődése a fénycső hulladék üveg por kiegészítővel

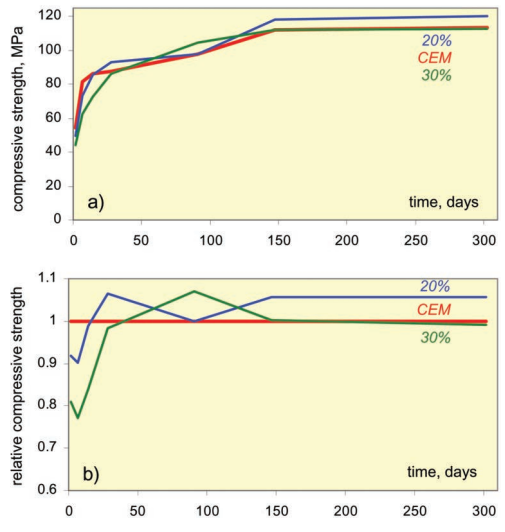

Fig. 10. Compressive strength and relative compressive strength of specimens with DRL WGP
10. ábra Nyomószilárdság és relatív nyomószilárdság fejlődése a villanykörte hulladék üveg por kiegészítővel

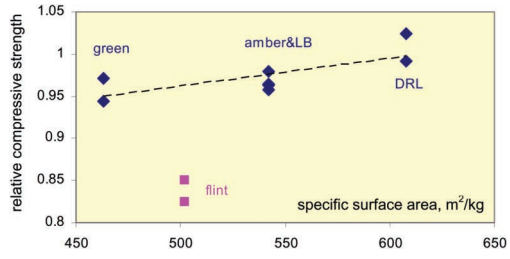

Fig. 11. Relative compressive strength vs. specific surface area of WGPs
11. ábra A relatív nyomószilárdság a hulladék üveg por kiegészítők fajlagos felületének függvényében

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Adorján **Borosnyói** – Patricija **Kara** – Lilla **Mlinárik** – Karina **Kaše**: *Performance of waste glass powder (WGP) supplementary cementitious material (SCM) – Workability and compressive strength*
Építőanyag, 65. évf. 3. szám (2013), 90–94. p.
http://dx.doi.org/10.14382/epitoanyag-jsbcm.2013.17

Megőrölt hulladék üveg (WGP) cement kiegészítő anyag (SCM) tulajdonságai – Bedolgozhatóság és nyomószilárdság vizsgálata

A megőrölt hulladék üveg cement kiegészítő anyagként történő felhasználása gazdasági és ökológiai haszonnal is jár: csökkenthető a hulladékdepóniákban elhelyezett anyag mennyisége, csökkenthető a nem megújuló nyersanyag felhasználás mértéke, csökkenthető a cementgyártás energiaigénye (kevesebb cement gyártása szükséges), csökkenthető ez által az üvegházhatást okozó gázok kibocsátásának mennyisége. Egy laboratóriumi vizsgálatsorozat keretein belül megőrölt hulladék üveget alkalmaztunk cement kiegészítő anyagként. A kiegészítő anyag mennyiség 20% és 30% volt a cement tömegére vonatkoztatva. Kimutattuk, hogy a megőrölt hulladék üveg cement kiegészítő anyag javítja a cementpép bedolgozhatóságát és közreműködik a megszilárdult pép nyomószilárdságában. Megfigyeltük, hogy a megőrölt hulladék üveg fajlagos felületének nagyobb szerepe van a kiegészítő anyagként történő hatékony működésben, mint a kémiai összetételének.
Kulcsszavak: újrahasznosítás, hulladék üveg, cement kiegészítő anyag, bedolgozhatóság, nyomószilárdság

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[6] Mohamed, K. R. – El-Rashidy, Z. M. – Salama, A. A.: In vitro properties of nano-hydroxyapatite/chitosan biocomposites. *Ceramics International.* 37(8), December 2011, pp. 3265–3271, http://dx.doi.org/10.1016/j.ceramint.2011.05.121
Books:
[6] Mehta, P. K. – Monteiro, P. J. M.: Concrete. Microstructure, properties, and materials. *McGraw-Hill*, 2006, 659 p.

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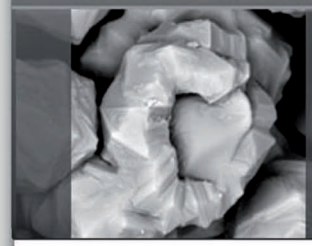

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Topics of is-icbm1

Event **is-icbm1** the **1st International Symposium on Innovative Carbon Based Materials** is focused but not limited on the following topics:

Carbides
Carbons
Composites
Diamonds
Fabrication and Applications
Fullerenes
Graphene
Graphite
Properties
Testing Methods, Equipment and Errors

The **aims** of the 1st International Symposium on Innovative Construction Materials are to distribute new results in research and development of construction and structural materials like ceramics, concretes, composites, metals, polymers and woods, as well as connect scientist, researchers, engineers and technologists of different universities, research institutes and organizations from different countries and continents.

Topics of is-icm1

Event is-icm1 the **1st International Symposium on Innovative Construction Materials** is focused but not limited on the following topics:

Asphalts
Binders and Adhesives
Concretes
Ceramics
Composites
Damages and Fractures in Construction
Materials
Geo-polymers
Glasses
Hybrid Construction Materials
Insulating Materials
Metals and Alloys
Mortars and Masonry Materials
Other Structural materials
Plastics and Polymers
Wood and Timber
Testing Methods, Equipment and Errors

Symposiums is-icbm-1 and is-icm1 are sponsored by several universities, scientific organizations, societies and associations around the World.

The Gold Sponsors

Anton Paar Gmbh (www.anton-paar.com), Department of Ceramics and Silicate Engineering, University of Miskolc (www.uni-miskolc.hu), Journal építőanyag (www.epitoanyag.org.hu), National Research Tomsk Polytechnic University (www.tpu.ru), IGREX Engineering Service Ltd, Journal cfi – ceramic forum international (www.cfi.de)

Symposiums is-icbm-1 and is-icm1 are organized parallel to the **ic-ic-cmtp3 — The 3rd International Conference on Competitive Materials and Technology Processes**. Registration, abstract submission and payment are available through ic-cmtp3 conference website of www.ic-cmtp3.eu

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The wonderful palace hotel of Hunguest Hotel Palota is in the exceptionally beautiful Bükk Mountains.

CEOS.fr

Comportement et Evaluation des Ouvrages Spéciaux vis-ŕ-vis de la Fissuration et du Retrait
(Behaviour and assessment of special construction works concerning cracking and shrinkage)
http://www.ceosfr.org

Within the CEOS.fr research project, several experiments on massive concrete structures were conducted in order to improve the knowledge on the cracking phenomenon and, at last, to provide the designers with reliable crack design codes, able to give a reasonable appreciation of crack width and spacing for theses structures as well as for classical ones.
CEOS.fr general objective is to make a significant step forward in the engineering capabilities for assessing crack patterns of concrete structures and predicting the expected pattern under anticipated design conditions. This general objective is detailed in three areas, corresponding to three different types of physical phenomena to be considered and modelled: 1) Cracking under monotonic loading, 2) Thermo-hydro-mechanical behaviour, 3) Seismic and cyclic loads.

ConCrack.org

ConCrack is an international benchmark for Control of Cracking in reinforced concrete structures.
http://www.concrack.org

The programme is the framework set up by French professionals involved in Building and Civil Engineering works to make a decisive step in the predictive capacity of models to describe the states of cracking, with the final objective to develop tools for designers and to give grounds to the evolution of standards. Based on the experiment driven from CEOS.fr, an international benchmark is organised dealing with the modelling of the behaviour of the tested mock-ups (monotonic and cyclic loading – prevented or free shrinkage).

Events calendar:

• CEOS.fr/ConCrack Workshop, Paris, December 10-12, 2009
http://www.ceosfr.org/international_workshop.htm
• ConCrack2 Workshop, Paris, June 20-22, 2011
• ConCrack3 Workshop, Paris, March 15-16, 2012
http://concrack3.ifsttar.fr
• ConCrack4 Workshop, Ispra, Italy, March 20-21, 2014

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Further information:
http://www.ceosfr.org
http://www.concrack.org
http://www.irex.asso.fr/

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