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International Conference on Civil Engineering,Infrastructure and Environment

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International Conference is the 8th international interdisciplinary peer reviewed conference which publishes works of the scientists as well as practitioners in the area where UBT is active in Education, Research and Development. The UBT aims to implement an integrated strategy to establish itself as an internationally competitive, research-intensive institution, committed to the transfer of knowledge and the provision of a world-class education to the most talented students from all backgrounds. It is delivering different courses in science, management and technology. This year we celebrate the 18th Years Anniversary. The main perspective of the conference is to connect scientists and practitioners from different disciplines in the same place and make them be aware of the recent advancements in different research fields, and provide them with a unique forum to share their experiences. It is also the place to support the new academic staff for doing research and publish their work in international standard level. This conference consists of sub conferences in different fields: - Management, Business and Economics - Humanities and Social Sciences (Law, Political Sciences, Media and Communications) - Computer Science and Information Systems - Mechatronics, Robotics, Energy and Systems Engineering - Architecture, Integrated Design, Spatial Planning, Civil Engineering and Infrastructure - Life Sciences and Technologies (Medicine, Nursing, Pharmaceutical Sciences, Physcology, Dentistry, and Food Science),- Art Disciplines (Integrated Design, Music, Fashion, and Art).

This conference is the major scientific event of the UBT. It is organizing annually and always in cooperation with the partner universities from the region and Europe. In this case as partner universities are: University of Tirana – Faculty of Economics, University of Korca. As professional partners in this conference are: Kosova Association for Control, Automation and Systems Engineering (KA – CASE), Kosova Association for Modeling and Simulation (KA – SIM), Quality Kosova, Kosova Association for Management. This conference is sponsored by EUROSIM - The European Association of Simulation. We have to thank all Authors, partners, sponsors and also the conference organizing team making this event a real international scientific event. This year we have more application, participants and publication than last year.

Congratulations!

Edmond Hajrizi,

Rector of UBT and Chair of IC - BTI 2019

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Compressive strength of precast concrete joints employing bulk mechanical connection devices

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Abstract. Dry assembled precast concrete joints are increasingly employed for fast and high-quality construction throughout the world. As a matter of fact, metal devices needed to connect the different elements often introduce a strong discontinuity in the current cross-section due to their size and to the presence of cavities needed to assemble the joint. Within the framework of a wider research project jointly carried out between Politecnico di Milano and DLC Consulting Srl on an innovative wall system, axial tests on a reference monolithic benchmark and three single joint specimens differing by reinforcement and/or mortar filling details have been carried out. A critical interpretation of the test results and of the compressive behaviour of the joints is provided by means of a numerical investigation through nonlinear static analyses performed with the software Abaqus.
Keywords: Precast structures, Compression testing, Numerical simulation, Joints, Mechanical connections.

Introduction

Precast structures are spread worldwide, and dry-assembled joints are increasingly employed for faster and higher quality construction [1,2]. As a matter of fact, the metallic mechanical devices needed to connect the different elements often become a strong discontinuity for the current cross-section due to their size and to the presence of cavities needed to assemble the joint [3]. These devices are usually conceived to ensure the correct flexural behaviour of the connected elements, being the axial strength often not relevant for typical columns of industrial frame buildings.

Recently, mechanical couplers have been proposed to be used in a dry-assembled precast wall system named Domus® conceived by DLC Consulting srl [4-5], later modified into Domus Dry® [6-7], where the wall element is lightened by inner vertical cavities, also according to different solutions recently proposed by other researchers [8-10]. The innovation brought by the type of wall investigated in this project was the driving force behind some previous full-scale tests carried out within the Safecast Project [11], which was aimed at characterizing the structural behaviour in bending [12], while the bearing capacity in shear and the shear-related failure modes have been recently investigated [13].

In this type of wall, the vertical joints are located at discrete positions, and in these positions, especially for medium rise buildings, the axial compressive strength of the joints can be crucial. In the technical literature there is no information about the influence of the bulk mechanical connection devices employed in these joints on the joint axial strength. Within the framework of a wider research project carried out jointly between Politecnico di Milano and DLC Consulting Srl of Milan on this innovative wall system, axial tests on a reference monolithic

benchmark and three single joint specimens have been carried out, showing an unexpected relevant axial strength loss of the mechanically jointed elements. This work is aimed at shedding light over this issue, by performing a numerical investigation through nonlinear static analyses performed with the software Abaqus [14].
A parametric study is performed to investigate the effects of the replacement of a concrete area with steel elements or cavities, considered separately. A model of the reference benchmark tested within the above-mentioned program is validated against the experimental results. Then, the three complex joints provided with mechanical connections differing by specific reinforcement or filling details have been modelled and subjected to static nonlinear analyses.
The difference found between experimental and numerical results has been then explained by further models including flaws of test setup and specimen assemblage procedure which have been identified following a critical review of the previous experimental activity.

Experimental tests

A total of 7 precast concrete elements were manufactured in a precast concrete production factory and shipped at the Laboratorio Prove Materiali, Strutture e Costruzioni (LPMSC) - Politecnico di Milano. These specimens consisted in a single monolithic element shown in Figure 1 and in three further specimens nominally identical made of top and bottom portions joined together in the laboratory prior to testing by activating the coupler mechanical connection and by pouring high strength non-shrinking mortar. Further description of the coupler device and experimental results are available in [15]. Excerpts from the technical drawings are provided in Figure 2 and details of the inserts are shown in Figure 3. All split specimens were provided, differently with respect to the solid monolithic specimen, with additional standard longitudinal and transverse (stirrups) reinforcement around the cavity in the top element and with high-strength steel baranchors and stirrups in the bottom element.
The split specimens differed among them from details of the mechanical joint: one specimen was provided with a simple coupler plate and filled with mortar only below it; a second specimen was provided with a coupler plate with arch-shaped additional reinforcement and filled with mortar only below it; a third specimen was provided with a coupler plate with arch-shaped additional reinforcement and completely filled with mortar.

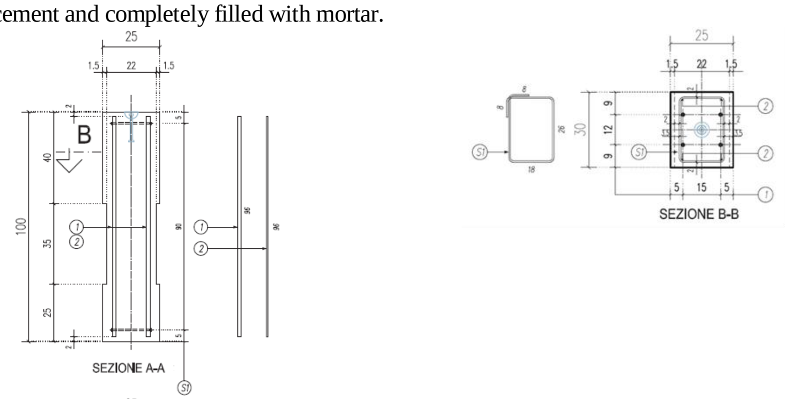


Fig. 1. Geometry and reinforcement layout of solid benchmark.

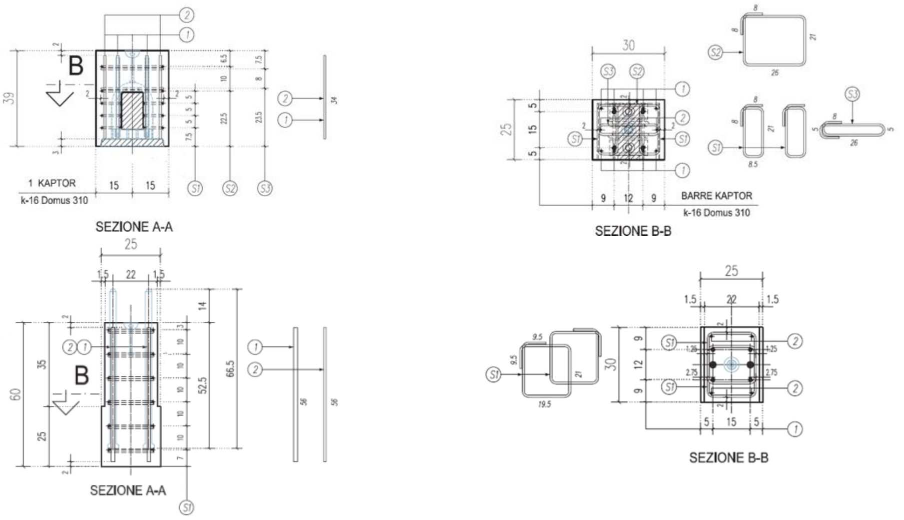
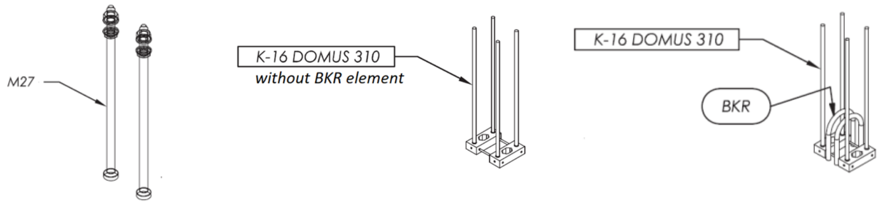


Fig. 2. Geometry and reinforcement layout of upper and lower elements of mechanical joints.



(a) (b) (c)

Fig. 3. Inserts for mechanical joints: (a) baranchors, (b) simple coupler, (c) coupler with additional reinforcement.

The above-cited four specimens were subjected to axial compression loading in a test rig employing an actuator with a capacity of 5000 kN reacting over two large-diameter threaded steel round columns. The top plate can be either fixed (no rotation allowed) or hinged. Figure 4 shows the specimens prior and after testing.

The test results, reported in the following in Table 1, provided an expected strength for the monolithic specimen and a dramatically decreased strength for all the specimens with mechanical devices, furthermore with unclear damage profiles exhibiting asymmetry.

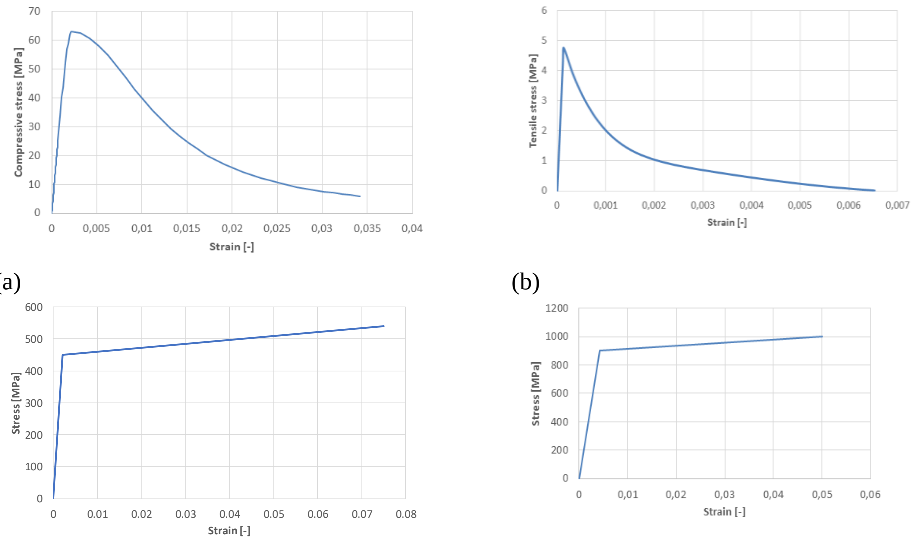
This unexpected result surprised the authors and pushed them towards a further development of the study on the subject, which was performed numerically.

(a) (b) (c) (d)

Fig. 4. Beginning (top) and end (bottom) of experimental tests on mockups: (a) additional cavity reinforcement and no mortar filling, (b) additional cavity reinforcement and mortar filling, (c) no additional cavity reinforcement and no mortar filling, (d) solid benchmark.

Numerical analysis

Initially, the solid benchmark was modelled in Abaqus and a static non-linear analysis was carried out to validate the modelling assumptions against the experimental results on this specimen. The assumptions on material properties are crucial for the accuracy of the model. In this study, the *concrete damage-plasticity* model was used for concrete, being it widely recognised proper for behaviour under primarily uniaxial and biaxial stress states [16,17,18,19]. The employed stress-strain relationships for concrete in compression and tension are plotted in Figures 5a and 5b. Despite a C45/55 class concrete was prescribed for the tested specimens, the model properties were calibrated on the basis of a mean cubic compressive resistance R_{cm} equal to 76 MPa, as from compression tests on standard cube specimens performed along with the experimental activity. The stress-strain diagram for steel is considered linear elastic-hardening with unlimited ductility. B450C and 10.9 steel grades were used for rebars and M27 bar anchors, respectively. The corresponding stress-strain curves are presented in Figures 5c and 5d. Similar models were set for the insert plate of S355 steel. Note that baranchors and plates were modelled for mechanical joints only (Figure 6).

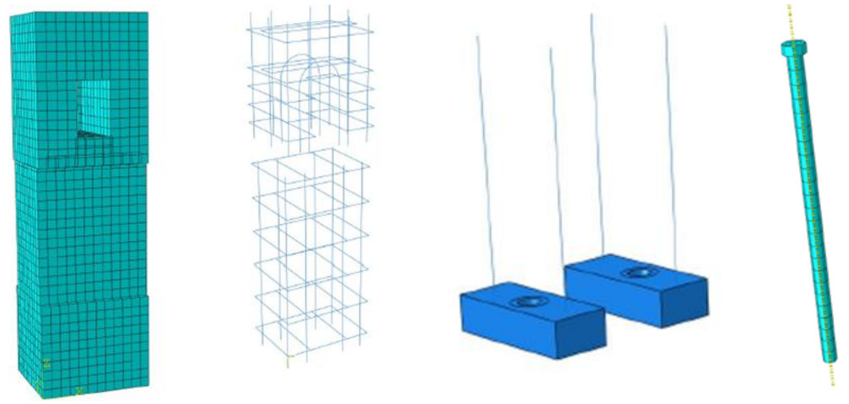


(a) (b)

(c) (d)

Fig. 5. Stress-strain relationships employed for: (a) concrete in compression, (b) concrete in tension, (c) rebar steel, (d) baranchor steel.

The overall element size was chosen equal to 25 mm. Hex elements were used, while sweep elements with medial axis algorithm were used for bar-anchor holes. Loading was applied by imposed axial displacement on top surface symmetrically. A displacement increasing with smooth amplification function with a final target displacement of 8 mm was applied, which allows to catch initial stiffness, ultimate capacity and softening branch of the joint behaviour. Difficulties were found in reaching convergence and stable analysis when static solvers were used. As suggested in literature [20,21], the *dynamic explicit solver* was selected due to its time control method in which the equation of motion is integrated in time using explicit central-difference rule. This is more efficient in solving discontinuities and contact problems. To perform a quasi-static analysis with this approach and to reduce the computational processing time and effort, the mass of the model was artificially increased provided its kinetic energy was maintained low throughout the analysis. This was achieved by means of the *semi-automatic mass scaling* option. To control the analysis whether it is quasi-static, internal energy and kinematic energy vs time figures are plotted at the end of the analysis and the differences are checked. According to [14], if the kinetic energy is less than 10% of the internal energy, the analysis is considered quasi-static.



(a) (b) (c) (d)

Fig. 6. Elements modelled: (a) concrete, (b) reinforcement cage, (c) coupler plates, (d) baranchor.

The outcome of the analysis on the solid benchmark provided the curves shown in Figure 7, where the maximum load (strength) was equal to 3780 kN. This number differs by only 3% with respect to the experimental result (3906 kN), and similar damage patterns were found between the numerical and experimental results. Hence, the model was considered reliable and the numerical analysis was brought further.

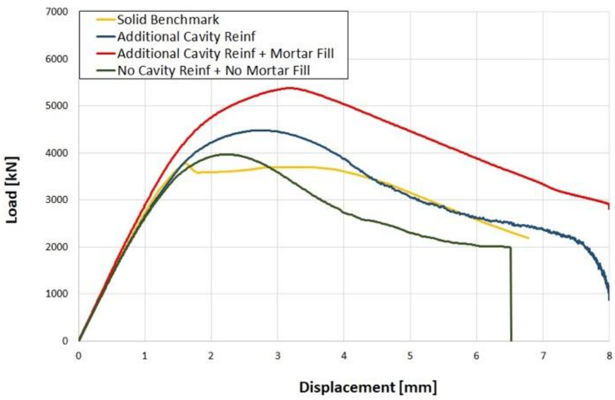
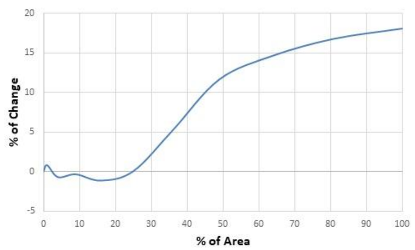
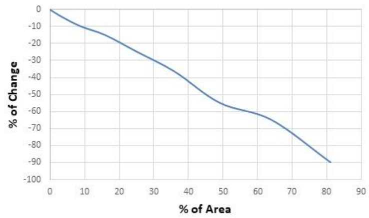


Fig. 7. Load vs vertical displacement (shortening) numerical curves of mockups.

A parametric study was carried out to investigate the effect of transversely placed steel inserts or cavities into the joint. The model of the solid benchmark was modified with transverse steel insert or cavities with increasing area starting symmetrically from the centre of the element. The added area of steel or cavity replaced concrete, and the thickness of this layer was kept constant at 30 mm. The results of parametric static non-linear analyses on the separate effects of steel inserts and cavities are shown in Figure 8. As expected, the strength decreases linearly with the area of the cavity (Figure 8b). On the other hand, replacing the concrete area with increasing steel area brings to a peculiar trend, with strength lowering in the range of 3 to 25% of steel area due to the local discontinuity effects, yet increasing afterward due to the higher mechanical properties of steel with respect to concrete. The strength reduction in the above-cited range is however estimated as being less than 2%, which is far from explaining the unexpected test outcomes on the mockups with mechanical connections.



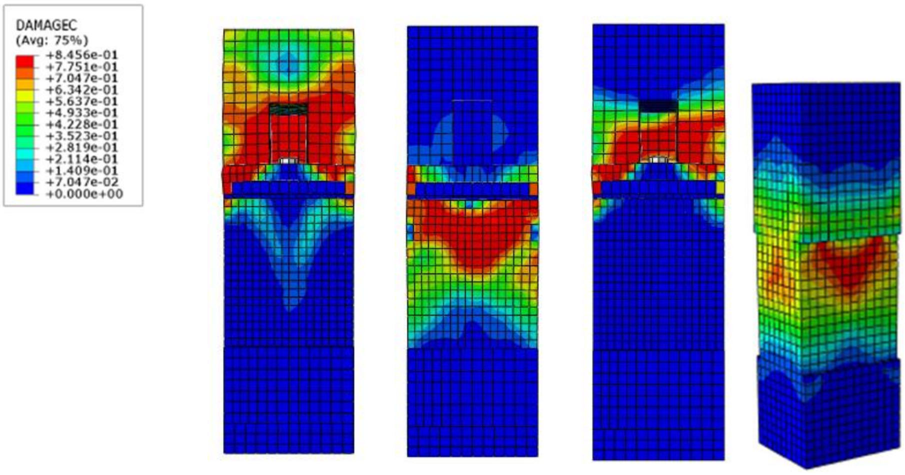
(a)



(b)

Fig. 8. Parametric analysis on the solid benchmark: (a) change in strength with replacement of concrete area with steel, (b) change in strength with replacement of concrete area with cavities.

The models of the mock-up joints with mechanical connections were then set and static non-linear analysis was carried out assuming perfect boundary conditions for them, too. High-strength mortar type Exocem G1 was used, whose mechanical parameters were evaluated experimentally through standard prism tests along with the experimental testing of the mock-ups. The average cubic strength was set to 80 MPa and the Young modulus at 27 GPa. An elastic model in compression was employed for the mortar, being verified that the mortar elements never overcame the limit of elastic behaviour set at 60% of the mean strength in the analyses. The results, in terms of load vs. displacement curves, are shown in the same Figure 7 cited above and the damage patterns in Figure 9. The joint described as (a), (b), and (c) in Figure 9 were found to be 18.6%, 42.4%, and 5.1% stronger than the solid benchmark, respectively. All of them differed from the solid benchmark for the additional mild reinforcement added at the contour of the cavity, and for the high-strength steel baranchor added in the central portion of the mock-ups, which contributed to the increase of strength. Mock-up (a) benefitted from the arch-shaped additional reinforcement of the plate, which explains its higher strength, and finally mock-up (b) further benefitted from the mortar filling of the cavity, providing the higher strength. The contribution of this additional reinforcement can be also recognised when extracting the maximum compressive stress in these elements, collected in Table 2, showing stress levels close or higher than the yielding. Critical damage was found to be located at the top part for mock-ups without complete mortar filling (Figure 9). Another beneficial effect for the mechanical joint mock-ups was provided by the transverse reinforcement (stirrups) placed along the element, which, creating an effective confinement effect, increased the concrete critical strength in the core (Figure 10).



(a) (b) (c) (d)

Fig. 9. Compressive damage distribution on mockups: (a) additional cavity reinforcement and no mortar filling, (b) additional cavity reinforcement and mortar filling, (c) no additional cavity reinforcement and no mortar filling, (d) solid benchmark.

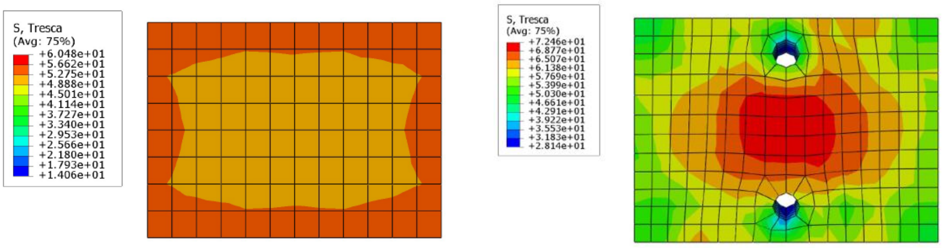
Table 1. Elastic stiffness and strength of mockups.

	Add. cavity reinf. + no mortar fill	Add. cavity reinf. + mortar fill	No cavity reinf. + no mortar fill	Solid benchmark
Numerical elastic stiffness - perfect boundary conditions [kN/mm]	2829	2998	2816	2865
Numerical strength - perfect boundary conditions [kN]	4486	5381	3973	3780
Numerical strength - vertical load on skewed specimens [kN]	3395	4090	3395	-
Numerical strength - inclined load on skewed specimens [kN]	2096	2331	1904	-
Experimental strength [kN]	2044	2425	2391	3906

Table 2. Maximum compressive stress in steel elements [MPa].

	Add. cavity reinf. + no mortar fill	Add. cavity reinf. + mortar fill	No cavity reinf. + no mortar fill	Solid benchmark

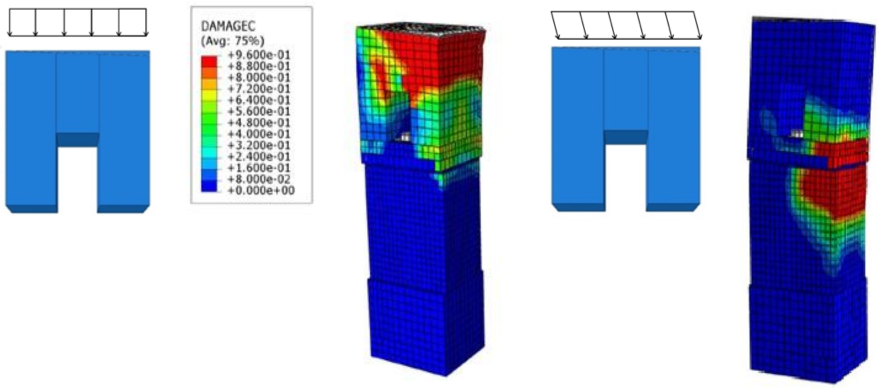
Longitudinal rebars	462	454	460	448
Baranchors	628	900	434	-
Steel plate	387	398	275	-



(a) (b)

Fig. 10. Effect of confinement: (a) weak confinement provided by longitudinal rebars in solid benchmark, (b) strong confinement provided by added transverse stirrups in mechanical joints.

The numerical results on the mock-ups with mechanical connections differed significantly from the experimental ones either in terms of strength and damage patterns. The strength values were all about half of that of the solid monolithic specimen and the damage patterns were highly asymmetrical. The tests have been critically analysed to understand the reasons behind this mismatching. It has been recalled that the assembled specimens were produced in such a way that a strong lack of planarity occurred, mainly due to the misalignment of the mechanical plates with the end counter-mold flat surface, and assembled without checking their planarity. To be noted that this issue was not found with the solid benchmark specimen, being it cast monolithically in a carefully assembled mould. The solution employed by the laboratory operators was to level the mockups with mechanical joints with a layer of sand-cement. The mechanical properties of this leveling layer were not experimentally investigated, but a research in the technical literature resulted in a dramatically lower strength and stiffness with respect to the mortar used to assemble the joints. This made the leveling layer more deformable than the mockup when subjected to the vertical load induced by the test rig, inducing concentration of vertical load on one edge of the specimen. Further static non-linear analyses were carried out including this layer with a plausible skewed upper surface, resulting into asymmetric concentration of damage (Figure 11a) and a relevant strength decrease of 15 to 25%, as presented in Table 1. This issue certainly contributed to the low experimental strength values, although it seemed not sufficient to thoroughly explain it. After a further check with the operators on the test rig, it was observed that the upper plate of the testing rig, provided with a central spherical hinge which could allow a rigid rotation of the plate, was not restrained against it. The presence of the softer skewed layer at the top of the mock-up could then not only brought to a localisation of the compressive action, but also to a rotation of the upper plate, progressively aligning with the original specimen skew, inclining the load, too. In tests with such high axial load, even very small inclination angles bring to dramatic increase of bending moment into the specimen, which further localises compressive strain and damage. Further numerical analyses carried out imposing contemporary increase of vertical displacement and rotation at the top of the mockups showed that, with a guessed ratio of 0.022 rad/mm, the strength dropped down to values similar to the experimental ones (about half the strength of the solid monolithic specimen), with highly asymmetric damage patterns, presented in Figure 11b for mockup (c), which qualitatively match with the experimental observations.



(a) (b)

Fig. 11. Different localisation of critical compressive damage for skewed specimen with weak mortar bed and cavity under: (a) vertical load (fixed rig), (b) inclined load (rotating rig).

Conclusion

From the numerical static non-linear analyses performed on solid benchmark and on mock-ups of equivalent mechanical joints, and from the observation of the experimental results and their critical review, the following conclusions can be drawn:

- The axial strength of the investigated joints provided with bulk mechanical connections is affected by metallic inserts, presence of cavities, additional cavity reinforcement, transverse reinforcement and mortar filling procedure;
- The numerical analysis provided sound interpretation of the mechanical/physical phenomenon under study and helped detecting the flaws occurred during the tests;
- Parametric analysis shows an almost linear decrease of joint compressive strength with the cavity area and a peculiar softly descending branch followed by a steep increase in strength after 25% of concrete replacement with steel;
- With respect to the solid monolithic element, the mechanical joint without additional reinforcement nor mortar fill had higher strength (+5%) due to higher longitudinal and transverse reinforcement, the one with additional reinforcement even higher (+18%) due to the arch-shaped bars compensating for the loss of concrete area in the region of the cavity, the mechanical joint mock-up with additional reinforcement and mortar fill showed the best performance (+42%) due to full mortar filling;
- The critical area, located in the recessed centre of the solid monolithic element, moved to the cavity surrounding for mechanical joints without mortar fill, and back to the centre for the mechanical joint with mortar fill;
- Planarity imperfection corrected with low strength mortar and free test rig rotation brought to a significantly reduced strength associated with a strongly asymmetric damage pattern, as confirmed by the numerical analyses;
- The analyses reflect an optimum configuration, while imperfections are always present in practice. However, the outcome of the work suggests that a proper planarity correction with high strength mortar would have avoided such problems;

- The adoption of cavity reinforcement and, especially, the full mortar filling of the joint are the techniques which ensure the best performance in compression.

Acknowledgements. The results presented in this paper are part of the results obtained in a comprehensive research project focused on an innovative dry-assembled precast system for residential buildings called Domus Dry® patented by DLC Consulting S.r.l. of Milan. This project was mainly financed by the Italian National Programme “Brevetti+” (2011) aimed at favouring industrial patents.

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Passive control of structures

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Abstract. Lately, powerful earthquakes stroke some parts of the world, while the Balkan peninsula was hit by moderate ones. During a powerful earthquake, a building structure is invaded by an enormous quantity of kinetic energy E_K. From the manner this energy is first absorbed, then dissipated throughout building structure depends, not only the reaction of structure, or structural elements in particular, but the nature, the distribution and the quantity of the damages also. As Nikola Tesla once quoted: "If you want to find the secrets of universe, think in terms of energy, frequency and vibration". In order to be able to achieve some degree of control, in structural engineering, the frequency is the fundamental parameter one must begin with. Passive control is actively implemented in the developed countries, whereas intensive laboratory examinations are underway the last two decades in the domain of semi-active and active structural control as well. This Paper, as a first deals with the static case, i.e. the behavior of a simple cantilever structure, treating its sensitivity towards shear and bending.

Keywords: Structural control, Energy, Base isolation, Seismic isolation

Introduction

When Nikola Tesla quoted: "*If you want to find the secrets of universe, think in terms of energy, frequency and vibration*", it is most certain he should have had more important things in his enlightened mind than the manner an engineering structure behaves when submitted to external actions, and yet, it is so meaningful for someone willing to understand how a structure behaves in this situation.

During the last hundred years and until today the design approach is the one based on strength of a structural element particularly or the whole structure. Nowadays, at the very heart of each of modern codes lies the design based on the interplay between the strength and ductility. Put simply: the ductility demand (*DD*) must be overcome by the ductility supply (*DS*), be it at the local or the global level.

Force-based methods, or as they will be called hereafter - conventional design methods or approach - impose as the basic requirement, that the structure responds passively to the hazards (earthquake, wind, etc.), mainly through the combination of resistance, on the one side, and deformability, energy absorption and dissipation, on the other. It is already well established that, during a strong earthquake, the structure undergoes significant deformation (and therefore damage) and, nevertheless, "survives" thanks to its inelastic "excursion" [1].

The designer, therefore, finds himself in situation where he/she has to choose between a strong structure, responding into the linear-elastic domain, i.e. suffering small if any deformations/damages at all, or, a weak one – undergoing important deformations/damages once the hazard has gone. The former requires big expenditures on primary lateral load resisting members, whilst the flexible one is economically much more suitable if built in such a way as to resist to moderate (frequent) hazards.

But what about a structure responding within velocity sensitive natural periods? Actual behavior of structures during strong earthquakes or winds has shown that neither of the design approaches mentioned above is enough in order to guarantee a satisfactory behavior – a new and modern approach, based on stiffness deployment is necessary. This paper in all its modesty aims to treat the subject of the so called "motion based" design. The approach uses some of fundamental mechanical principles in order to first absorb and afterwards dissipate a good part of the energy input imposed to a structure, fulfilling thereof two of the principal requirements: Collapse prevention and serviceability (normal use) including users comfort level.

Problem definition - conceptual design, creative phase and finally problem refining or carving is directly connected with human activity [2], whilst machine interaction can help the above-mentioned activities, but can never replace them.

This paper is a modest attempt to increase the awareness in relation to the nonconventional approach when undertaking the structural design of highly sensible civil engineering structures, namely high-rise buildings.

Human response and sensitivity to vibrations

Whereas conventional design of structures tailors its members based on strength requirements, establishes the relevant stiffness properties and only then checks the serviceability criteria (SLS – EN 1990), while maintaining the strength as the principal requirement (ULS), the ever increasing trend of designing flexible structures, shifts the emphasis towards displacement (motion) based design.

Frequently, some facilities, such as hospitals, data storage centers, etc., must remain operational even after they undergone a strong earthquake. Another example could be semi-conductor manufacturing center, where hypersensitive equipment must stay (almost) motion-free, since its monetary value may sometimes even exceed that of the building itself. On the other hand, comfort limits for humans are somewhere near $0.02g$ in terms of building accelerations. The parameters affecting human sensitivity to vibrations are enlisted excellently in [1 – Bachmann, 1997], whilst the Codes treating the subject are [ISO 2631] and [DIN 4150]. As an example, the human perceptibility threshold (person standing) for vertical harmonic vibrations is $34\,mm/s^2$ – just perceptible, to $1800\,mm/s^2$ – intolerable.

While sight or hearing are two sensory phenomena centered on two of the basic organs of the human body, oscillation receptors are like those of heat / cold and are in some degree a continuation of the nervous system. Thus, the human finger has receptors with such a degree of sensitivity, that it can probe oscillations whose amplitude revolves around values of $1 \cdot 1^{-3}\,mm\,to\ {}^{1}/_{20} \cdot 10^{-3}$ [1].

When a person works within a shaking skyscraper, he feels uncomfortable on a scale that can range from "barely sensitive" to "intolerable" one. The degree of comfortability depends a lot on user's location, as he will not feel the same when sitting in his office on the $52nd$ floor of a New York skyscraper or on the second floor of a restaurant in Berlin at an event organized by his friends.

Among the basic parameters that affect human susceptibility to oscillations are [3]: position (standing, sitting, lying down), direction of incidence with respect to the spine, personal activity (at rest, walking, running), sharing the activity with others, age and gender, frequency of occurrence and time of day, the character of the weakening (extinction) of the oscillations, etc., whilst the intensity of perception depends on displacement, velocity and acceleration amplitudes, duration and frequency of vibrations [3].

As for the *criteria* related to the intensity of perception [3] (sensitivity), they are expressed through a single parameter which is the *effective acceleration* (rms - Root Mean Square) and is given by expression (1.1) as follows:

$$a_{eff} = \left(\left({}^{1}/_{T} \right) \cdot \int_0^T a^2(t)dt \right)^{1/2} \quad (1.1),$$

Where T – is the time period within which effective acceleration has been measured.
ISO 2631, distinguishes three different levels of human inconvenience (comfortability) to vibrations:
The reduced comfort limit, which is the threshold at which human activities such as eating, reading or writing are hampered by vibrations.
The fatigue-decreased proficiency boundary, which refers to the threshold where repeated oscillations cause fatigue in (working) staff, with a direct (negative) result in reduced productivity. In intensity, this threshold corresponds to three times the limit of reduced comfort.
The *exposure limit* is the upper limit of oscillation tolerance for the health and safety of the individual. This limit corresponds to six times the limit of reduced comfort.
Additional Information Required by the Volume Editor

Sensitivity of a cantilever structure depending on type of action - shear load or bending moment

From classical beam bending theory [4], the differential equation governing the beam deflections is given by equation (2.1) below:

$$z'' = -\frac{M}{EI} \quad (2.1)$$

Where: z – vertical deflection; M – bending moment; E – elasticity modulus; I – moment of inertia of the beam cross section. In the case of a cantilevered beam (see figure below), deflections are given by the expression (2.2) [4],

$$z = z_M + z_T \quad (2.2)$$

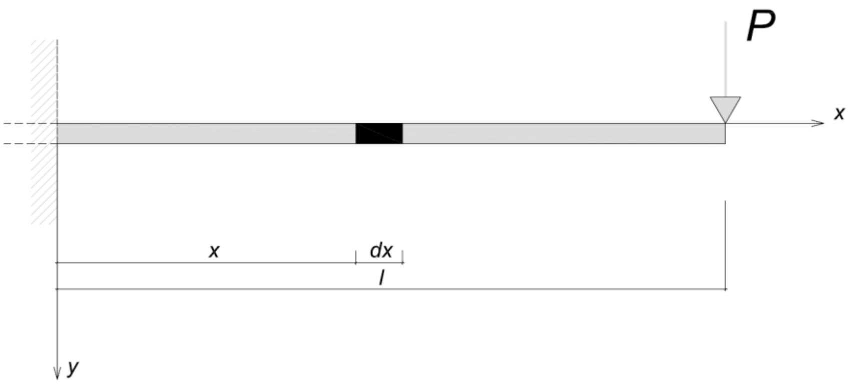
Where the displacement due to bending moments is given by expression (2.2a),

$$z_M = \frac{P}{EI} \cdot \left(\frac{l}{2} - \frac{x}{6} \right) \cdot x^2 \quad (2.2a)$$

Whilst the deflection due to the transversal (shear) loads is given by expression (2.2b),

$$z_T = \frac{P \cdot l}{GF} \cdot \alpha \quad (2.2b),$$

where: α – coefficient depending on the shape of cross-section; G – shear modulus; and, F – cross-section area of the beam.



$Fig.$ 2.1 Cantilevered beam submitted to a concentric load P

Timoshenko [4], gave an expression (2.3), which is like (2.2),

$$z = \frac{P \cdot l^3}{3EI} \cdot \left(1 + 0.98 \cdot \left(\frac{d}{l}\right)^2\right) \tag{2.3}$$

Where: d/l – represents the slenderness ratio of the beam.
Based on any of fundamental principle of mechanics, one can easily derive the expression for bending or shear stiffness of the beam (expressions 2.4), meanwhile, the fig. 2.2 below shows both bending and shear stiffness in function of beam's slenderness ratio d/l. It is worthy to remark, that for a slenderness of $d/l \sim 1.02$, the share between relative participation is $50\ \%$ approximately.

$$\begin{cases} k_{p\ddot{e}rkulje} = 3EI/l^3 \\ k_{prerje} = EF/3l \end{cases} \tag{2.4}$$

$Fig.$ 2.2 Percentage of participation of shear and bending on deflection for the cantilevered beam shown in $Fig.$ 1

It is clear, from the Fig 2.2 above, the degree of shear-stiffness "mobilization" towards deflection participation is from *low,* for flexible structures (high slenderness ratio, participation ratio $\sim max$ 4%) to *very low*, for "bulky" structures (low slenderness ratio $\sim 0\%$). This speaks a lot about the degree of sensitivity of a structure, when the slenderness is taken as a comparative measure.

Static effect cantilever beam with high bending stiffness (elevated sensitivity towards the effect of shear loads)

Let us consider, once again, the cantilevered structure in Fig 1 above, but rotated anticlockwise for 90 degrees now, submitted to a horizontal load P.
Shear stress due to the above loading conditions is given by expression (3.1) below,

$$\tau_{pr} = {}^{P}\!/_{F_{pr}} \tag{3.1}$$

Where: F_{pr} – represents the area cross section of the beam within which shear stresses are assumed to be constant (the distribution is parabolic!)
In order to comply with the *resistance design criteria* (*ULS*) of the cross section, the necessary cross-sectional area of the beam must fulfil the requirement according to the expression (3.2) below,

$$F_{pr}^{rezist} \geq P / \tau_{pr}^{lej} \tag{3.2}$$

Where: τ_{pr}^{lej} - is the admissible shear stress for the selected material.

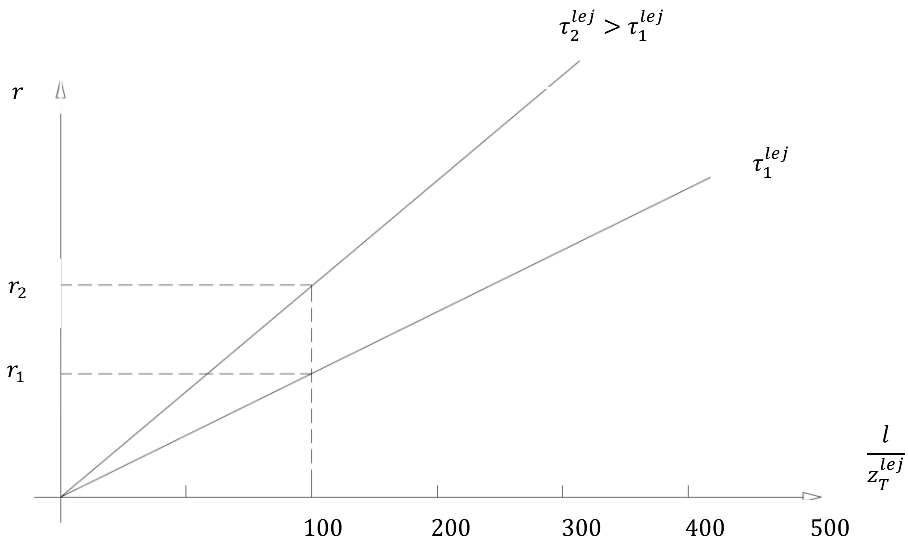
In the same way, the necessary cross-sectional area of the beam in order to comply with *admissible deflections criteria (SLS - serviceability)*, must fulfil the requirement according to the expression (3.3) below,

$$F_{pr}^{shfrytzueshm.} \geq \frac{P}{G} \cdot \frac{l}{z_T^{lej}} \tag{3.3}$$

Where: z_T^{lej} – represents the admissible (acceptable) displacement of the tip of the cantilevered structure – normally given in advance, in accordance with user's comfort [3].
Let now build the ratio between the two cross-sectional areas given by expressions (3.2) and (3.3), see expression (3.4) below,

$$\boldsymbol{r}_1 = \frac{F_{pr}^{shfrytzueshm.}}{F_{pr}^{rezist}} = \frac{\tau_{pr}^{lej}}{G} \cdot \frac{l}{z_T^{lej}} \tag{3.4}$$

The ratio $\boldsymbol{r}_1$ represents the threshold which underlines the relative importance of the *displacement design constrains* versus resistance *(strength) design constrains*.
The Fig 3 below shows the relation between $\boldsymbol{r}_1$ and l/z_T^{lej}, for given values of $\tau_{pr}^{lej}\ /G$, which is constant for a selected material (e.g. steel $S235$). Therefore, the ratio $\boldsymbol{r}_1$ grows linearly, so for decreased values of allowed deflections z_T^{lej} it grows continuously and thus it puts added emphasis over displacements (on motions).
Also, from the equation (3.4), we can see that if we attempt to "intervene" in the quality of the material, it is clear the ratio $\boldsymbol{r}_1$ increases ($r_2 > r_1$), which practically means yet more sensitivity (increase of structural sensitivity).



$Fig.$ 3.1 Graphical presentation of sensitivity $\boldsymbol{r}$, for the cantilevered structure in function of its slenderness l/z_T^{lej}

Starting from the beginning of the 20th century, and then continuing into the forties until its end, the technology of materials used in civil engineering has been under a linear increase - both in production procedures, increasing their quality, and especially their mechanical resistance refinement. It is particularly noteworthy, that while the mechanical resistance (e.g. concrete or steel) has been doubled, at least, if not quadrupled in some cases, their material stiffness (corresponding modulus of elasticity) has remained almost constant [2].

Static effect cantilever bending beam with low shear bending (elevated sensitivity towards the effect of bending loads)

Let analyze once again the cantilevered structure as shown in Fig 2.1. The bending moment at cantilever's spring (the fixed support) is

$$M = -P \cdot l \quad (4.1)$$

The bending stress σ is a well-known expression from the Strength of materials

$$\sigma_{përk} = {}^{M}/_{I_{pr}} \cdot z \quad (4.2),$$

Or, if expressed in terms of section modulus $W_{për}$

$$\sigma_{përk} = {}^{M}/_{W_{për}} \quad (4.3),$$

Where: I_{pr}- is the moment of inertia of the cross-section, z – is the fiber's distance from the neutral axis, $W_{p\ddot{e}r} = I_{pr}/\left({}^{d}/_{2}\right)$ – is the section modulus

The displacement at the tip of the cantilever, under the actual load is

$$u_{p\ddot{e}rk} = {}^{P \cdot l^3}\big/_{3EI_{pr}} \qquad (4.4),$$

In order to comply with the *resistance design criteria* (*ULS*) of the cross section, the necessary cross-sectional moment of inertia of the beam must fulfil the requirement according to the expression (4.4) below,

$$I_{p\ddot{e}rk}^{rezist} \geq {}^{P \cdot l \cdot d}\big/_{2\sigma_{p\ddot{e}rk}^{lej}} \qquad (4.4)$$

Where: $\sigma_{p\ddot{e}rk}^{lej}$ - is the admissible bending stress for the selected material.

In the same way, the necessary moment of inertia of the beam in order to comply with *admissible deflections criteria (SLS - serviceability)*, must fulfil the requirement according to the expression (4.5) below,

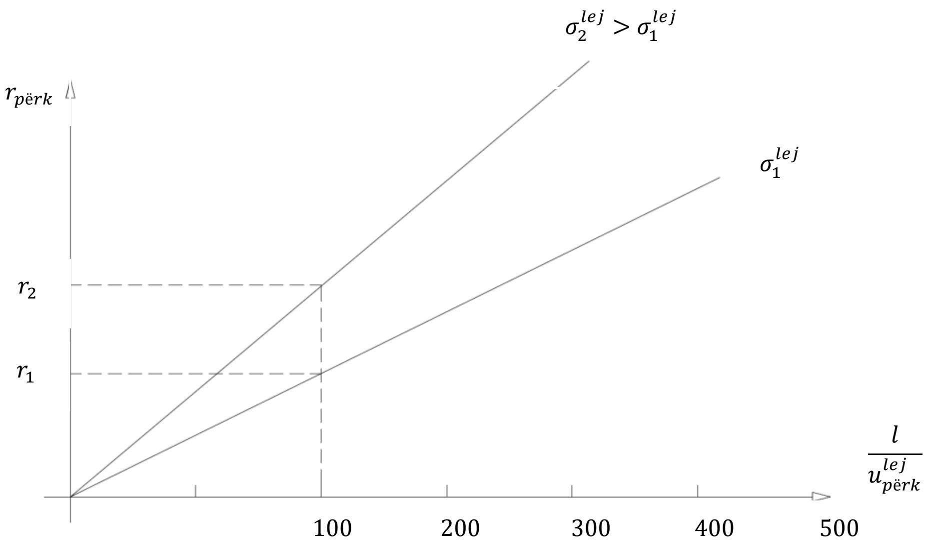
$$I_{p\ddot{e}rk}^{shfrytzueshm.} \geq {}^{P \cdot l^3}\big/_{3Eu_{p\ddot{e}rk}^{lej}} \qquad (4.5)$$

Where: $u_{p\ddot{e}rk}^{lej}$ – represents the admissible (acceptable) displacement of the cantilever's tip.

Once again, we establish the ratio between the moment of inertia required to satisfy *serviceability criteria* to the moment of inertia required to satisfy *strength criteria*

$$r_{p\ddot{e}rk} = \frac{I_{p\ddot{e}rk}^{shfrytzueshm.}}{I_{p\ddot{e}rk}^{rezist}} = \frac{P \cdot l^3}{3Eu_{p\ddot{e}rk}^{lej}} \cdot \frac{2\sigma_{p\ddot{e}rk}^{lej}}{P \cdot l \cdot d} = \frac{2l}{3d} \cdot \frac{\sigma_{p\ddot{e}rk}^{lej}}{E} \cdot \frac{l}{u_{p\ddot{e}rk}^{lej}} \qquad (4.6)$$

Like the Fig 3.1, the plot below shows the dependence of the ratio $r_{p\ddot{e}rk}$ in function to mainly three parameters: *global slenderness* $\frac{l}{d}$, *allowable deformations* $\frac{\sigma_{p\ddot{e}rk}^{lej}}{E}$, and finally the ratio of the beam's span l to allowable tip displacement $u_{p\ddot{e}rk}^{lej}$.



$Fig.$ 4.1 Graphical presentation of sensitivity parameter $\boldsymbol{r}$, for the cantilevered structure in function of its slenderness $l/u_{përk}^{lej}$

Like in the case of the shear beam, each increase of $l/u_{përk}^{lej}$, i.e. the decrease of the allowable displacement $u_{përk}^{lej}$, puts more emphasis on displacement if span is to l remain constant. One could increase the allowable bending stress (steel grade or concrete class), hoping to decrease the (overall) sensitivity, but $\sigma_{përk}^{lej}$ puts even more emphasis on *displacement constraint*, as it is shown in the $Fig.$ 4.1 above.

For example, let consider a steel beam of strength class $S235$, with allowable stress (yield strength) $f_{y,k} = 200\,N/mm^2$ [5], a Young's modulus $E = 170000\,N/mm^2$, and a slenderness $l/d = 8$. The value $l/u_{përk}^{lej}$ at which (the sensitivity) a transition from strength to serviceability occurs can easily be calculated from expression (4.6) ($r_{përk} = 1$),

$$\left.\frac{l}{u_{përk}^{lej}}\right|_{r_{përk}=1} = \frac{3}{2}\cdot\frac{d}{l}\cdot\frac{E}{\sigma_{përk}^{lej}} = \frac{3}{2}\cdot 8^{-1}\cdot\frac{170000\,N/mm^2}{200\,N/mm^2} = \sim 160$$

Thus, for $\frac{l}{u_{përk}^{lej}} > 160$, i.e. $r_{përk} > 1$, the structural design of the cantilevered structure is governed by its tip displacements.

Let now try to improve the steel grade and instead of $S235$ we use steel $S355$, with $f_{y,k} = 355\,N/mm^2$, whilst Young modulus and slenderness remains unchanged,

$\left.\frac{l}{u_{përk}^{lej}}\right|_{r_{përk}=1} = \frac{3}{2}\cdot\frac{d}{l}\cdot\frac{E}{\sigma_{përk}^{lej}} = \frac{3}{2}\cdot 8^{-1}\cdot\frac{170000 N/mm^2}{355 N/mm^2} = \sim 90$, so it is evident now, that displacement controls the Design process, for the full range of the admissible displacements $u_{përk}^{lej}$.

Summary

The last decades, many research studies have been going on relating to the Design approach. Currently, most structural codes worldwide have adopted the approach based on force as a design strategy, i.e., an approach based on giving the necessary strength/ductility to the structural elements, or to the whole structure in general.

Now, in a philosophical point of view – does it exist an objective reason of the force to exist, and how do we cognitively recognize it? It is a generalized displacement of a node, that makes us knowledgeable of the force existence, that is, because of the fact we see the displacement, we are certain of the force existence. It is precisely this fact, although known since the dawn of engineering, that during the last three decades initialized the displacement design approach thinking within the professional community, first in USA, and afterwards elsewhere in industrialized countries.

Human being does possess a sensitivity towards external natural phenomena in general, and vibrations in particular. Thus, acceleration of the order 0.02g are the threshold at which humans begin to feel uncomfortable [Eurocode 8]. On the other hand, structures, in dependence of their physical characteristics, do possess a certain level of sensitivity. A structural designer, when has several possibilities at his disposal: to design a strong structure, that is, a structure responding quasi statically; a structure designed in the domain of resistance/ductility response; a flexible to very a flexible structure, responding within the increased displacements domain. The first family of structures requires higher initial costs, the second one can be economical, whilst the last family can be built with medium to low initial costs but can suffer important to very high damages after it has been submitted to externa hazards.

In this first paper, hoping to be continued with yet another one, the Author has attempted in a modest yet significant manner to underline the importance of structural sensitivity, first for a shear beam and second for a bending beam. For the first family of structures the importance of shear stresses and their contribution to the total amount of displacement has been treated, based on Timoshenko's classical beam theory [Timoshenko], whilst in the second case, the bending stress importance for the same parameter has been analyzed. Both for the first as well as for the second case sensitivity parameter $\boldsymbol{r}$ [6] has been represented graphically, in order to underline the importance of *serviceability criteria* towards the *strength* (resistance) *criteria*.

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COMPARATIVE DESIGN ASPECTS OF REINFORCED CONCRETE LIQUID RETAINING STRUCTURES

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Abstract. The Reinforced concrete liquid retaining structures must be designed so that the facility to be safe from leakage. One of the determining factors for the design of reinforced concrete liquid retaining structures is design with restriction of cracks. According to EN 1992-3 are defined four tightness classes in order to show degree of protection against leakage in design of reinforced concrete liquid retaining structures. In general, design of reinforced concrete liquid retaining structures can be done in two main cases: design without cracks and design with induced cracks which are controlled one. In this paper is shown the case where in design are foreseen to apply the concept of controlled cracks by application of joint tube for induction of cracks which takes rheological effects of concrete. The rheological effects of concrete can cause cracks which on one side are very difficult to predict where they will appear, while on the other side the appearance of these cracks greatly affects the degradation of the structure. Joint tube for induction of cracks is applied to eliminate these effects. The practical application of this concept was done during the construction of the facilities of a drinking water treatment plant in our country and it was concluded that exactly the purpose for which they were applied was achieved. This concept is foreseen also by EN 1992-3. The benefits of applying this concept are numerous and the most important is the safety that is achieved against leakage.

Keywords: Reinforced concrete liquid retaining structures; Controlled cracks, Induced contraction joint; Limitation of cracks; Protection from leakage; Tightness classes.

Introduction

Proper design of reinforced concrete liquid retaining structures by which design is achieved to ensure the structure from leakage is definitely a challenge in construction engineering. In this case, the mechanical aspects of the actions on structures and the quality aspects of the materials to be used must be taken into account in the design. The application of innovative methods of design and construction of these structures has been found to be more efficient than standard classical methods. Among the most efficient methods is controlled crack design which is achieved through the use of induced cracks in certain concrete sections of the structure's elements.

Design principles of reinforced concrete liquid retaining structures

In general, the design of reinforced concrete liquid retaining structures can be done in two main concepts:

1. Design of structures with cracks limitation or without cracks, whereby the purpose of cracking restriction is achieved by the acquisition of the dimensions of the structure elements on the one side and the application of certain forms and amounts of reinforcement on the other, as recommended with design codes, in this case with EN 1992, and
2. With the application of induced contraction joints where the cross-section of the concrete element is deliberately reduced in order to cause cracking at the desired position of the structural element of the structure.

This paper aims to present the advantages of applying the second case, namely causing cracks that can be controlled.

The design case with controlled crack design is included also in EN 1992-3 where Annex N and paragraph 9.6.6 provide recommendations with measures to be taken into account when applying this concept.

Having in consideration of the need for leak protection and in order to apply this concept when applying controlled crack, different manufacturers have issued different products with the application of which this goal can be achieved.

Practical application of the design with controlled cracks

This paper will present the practical application of design with controlled cracks in one project of water retaining facilities implemented in Kosovo. The water retaining facilities where this concept was used were facilities of a drinking water treatment plant and therefore we had to adopt a solution that ensured that there was no leakage from the walls of the facilities (the leakage had to be prevented either for the outflow of water from the facility or even for the outflow of water into the facility from outside which in this case was the soil behind the walls).

After analyzing the possible solutions, we have come to the conclusion that the most efficient solution in our case is design with controlled cracks. For this purpose a joint tube for induction of cracks is provided. This tube is placed in the middle of the wall while in the two sides of the wall before concrete pouring are mounted triangular shaped wooden bars to leave a space which after removal of the formworks are filled with waterproof mortar, this waterproof mortar has flexible mechanical properties.

In the case of the flucctuation of creep and shrinkage of concrete and also flucctuation of loads in structural elements (walls) than joint tube for induction of cracks and insulation grout in gaps has enought flexibility capacity to accept these deformations without suffering damage that may cause leakage, so even after all kinds of potential impacts these imposed controlled cracks remain functional.

The joint tube for induction of cracks consists of thermoplastic PVC material which is weldable to other water-stop bars or tapes thus in combination forming a closed loop waterproofing. The tube has six ribs which have some teeth-shaped parts, so the geometric shape of the tube and its ribs enables a good bond with the surrounding concrete. As shown in detail, the pipe has two ribs that have no teeth, and these two ribs should be positioned normal to the outer surfaces of the wall. The material of which the pipe is made has a slight increase in volume from the chemical reaction when it comes into contact with the wet concrete providing even more bonding between the pipe and the concrete. These tubes are usually fitted with a U-PVC material pipe inside to hold the pipe in proper position.

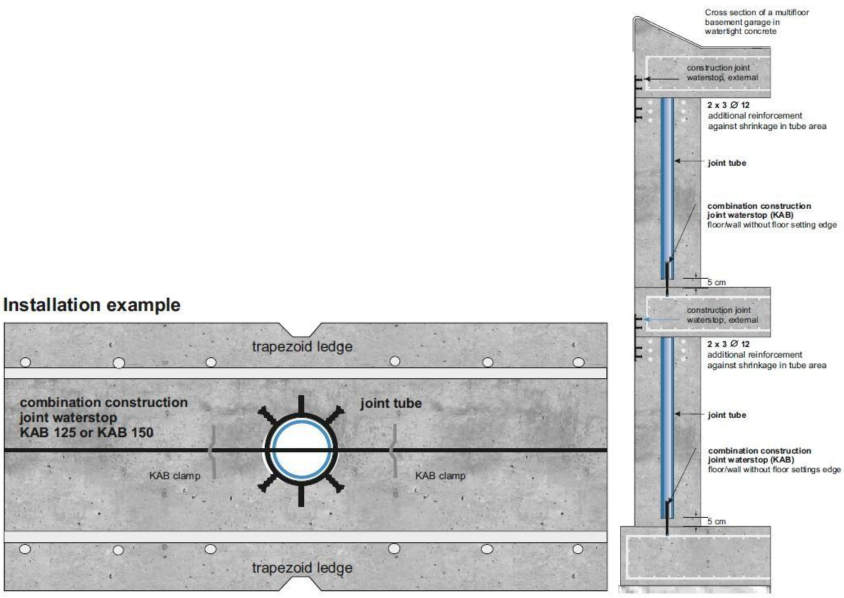

Figure 1: Detail for the placement of the tube in concrete wall

Figure 2: Images from practical application of joint tubes for induction of cracks in concrete wall

The principle of operation of this concept of controlled crack is based on obtaining the rheological effects (shrinkage and creep) of concrete from the joint tube for induction of cracks which is very well bonded to the concrete and thanks to the very good properties of the pipe it accepts all impacts without transmitting them to the concrete.

In the above mentioned project the geometric shapes of the facilities have been rectangular. In rectangular shaped facilities it is easier to determine the proper position for pipe placement.

The proper position for tube placement is assumed where the stiffness of the structural element (in this case wall) or the stiffness of the structure is the smallest which of course coincides with the position where the impacts from the concrete shrinkage are maximal as well, and also static impacts are maximal.

In fact, the concept of applying the joint tube for induction of cracks is based on the cracking caused by the reduction of the cross-section of the structural element, which inevitably orients the development of concrete shrinkage in this position.

Depending on the calculated values of the shrinkage in concrete, the tube of adequate size may be selected.
Because the effect of the shrinkage of concrete is maximal at the position where the element (wall) stiffness is smaller and this stiffness is further reduced due to tube placement, also at this position mainly the static impacts are maximal on the opened buildings such as water retaining structures, then it is advisable to analyze the wall in calculation as a cantilever which requires larger dimensions but this is usually not a problem for water retaining structures which, due to other aspects, also acquire larger dimensions. While in buildings where walls have a statical system of bottom and top connection (such as basement walls, slabs, etc.) generally no increase in element thickness is required.

Advantage of the use of joint tube for induction of cracks

Selection of the design of reinforced concrete liquid retaining structures with controlled cracks has several advantages compared to the design with cracks limitation or without cracks, including:

1. Provides complete safety to eliminate leakage,
2. Cracks are controlled and the possibility of cracks appearance in other positions of the structural elements is eliminated,
3. It is a very economical solution (smaller element thickness and smaller amount of reinforcement),
4. Easy to apply and maintain, etc.

When applying this concept, appropriate measures must be taken to fully achieve its intended purpose by applying the recommendations given in EN 1992-3, among others:

- Must have proper sealant material selected for filling gaps at induced crack depending on the liquid to be retained,
- Sealants to joints must be constructed in such a way as to enable inspection, maintain and easy repair or renovation of them.

Conclusions

With the application of joint tubes for induction of joints in the abovementioned facilities no cracks have been appeared in any other position on the walls of the buildings, ie the effect of the shrinkage of the concrete has been successfully taken by tube, thus ensuring no leakage of facilities, respectively is reached the purpose of their application.
Also, at the pipe contact with the water-stop tapes there is no leakage which means that even at these positions waterproofing has been fully achieved.
As the concrete begins to harden, cracks begin to appear at the place where the pipe is laid and passing the time this crack is increased to a normal expected level. Then the left joint is filled with the recommended flexible mortar and then the facility is tested for leakage in which no leakage is shown to the positions where the joint tube for induction of cracks was used. In this project, the joint tube for induction of cracks was also used in horizontal position on the cover plate of the accumulation tank due to the large amount of concrete that this plate had, and also in this case it was achieved to eliminate the effect of concrete shrinkage and not allowing cracks to appear in unwanted positions.
From the lessons we have learned from the practical use of this concept in the above mentioned project as well as at other buildings, we recommend applying it to basement floor of buildings or at other buildings in contact with wet ambient as it is a safe concept for eliminating the

shrinkage effect in concrete which, if not treated with enough care can cause severe damage which are very difficult and very costly to repair.

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On the identification of the axial force in stay cables with unknown boundary conditions

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Abstract. Identification of tensile force in axially-loaded structural elements is of paramount importance for health monitoring and safety assessment purposes. Dynamic testing techniques provide the ground for quick and cheap identification strategies, based on the knowledge of: (a) a set of identified natural frequencies, and (b) a structural model that relates natural frequencies to the axial force value. Reliability of results, hence, is inherently related to the predictive capabilities of the underlying structural model. Errors may arise, in particular, from the modeling of boundary conditions. The present paper analytically investigates the effect of unknown boundary conditions on the modal properties of a shallow cable with small bending stiffness. Starting from theoretical results obtained on this archetypal structural model, a simple but effective numerical procedure to identify the axial force in stay cables is then presented.

Keywords: Stay cables, Axial-force identification, Health-monitoring, Safety assessment, Rotational stiffness.

Introduction

Stay cables are lightweight and lightly damped structural elements, whose transverse vibrations can be easily set up by providing relatively small amounts of input energy. Dynamic testing techniques based on ambient vibration or hammer impact tests, hence, can be effectively used to get estimates of the lowest natural frequencies of a stay cable.
Knowledge of a set of measured natural frequencies along with a suitable structural model updating strategy, then, can be used to identify the cable axial force value (see e.g. [1]). Within this framework, reliability of the identified axial force values depends, among the other factors, on the predictive capabilities of the underlying structural model.

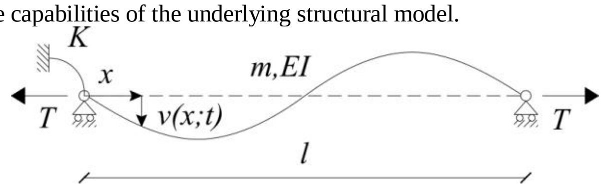


Fig. 1. Schematic representation of a stay cable subject to a tensile load T.

Typical models of the literature are based on the assumption of ideal boundary conditions, in the form of either perfectly hinged or perfectly clamped cable end sections (see e.g. [1, 4, 7-9]). A more realistic structural scheme could be defined, however, by considering equivalent translational and rotational springs at the cable end sections to model the flexibility of both the anchoring devices and the support structures (e.g. deck and tower for cables in stayed bridges). Proper definition of equivalent springs strongly depends on the particular technology adopted to realize the cable anchorages (see e.g. [2]) and is inherently related to several different sources of uncertainties, such as those related to geometric imperfections and aging of the anchoring devices.

The present paper presents an analytical investigation of the effects of unknown boundary conditions on the modal properties of a shallow cable with small bending stiffness. Starting from theoretical results obtained on this archetypal structural model, a simple but effective numerical procedure to identify axial force in stay cables is then presented and validated through an extensive numerical testing campaign.

Formulation of the problem

Let us consider a stay cable of length l, with constant bending stiffness (EI) and mass per unit of length (m), subject to an axial force T (see Fig. 1). By neglecting sag- extensibility and shear deformability effects, undamped planar flexural vibrations are governed by the partial differential equation:

$$EI\, \partial^4_x v - T\, \partial^2_x v + m\, \partial^2_t v = 0, \qquad (\mathbf{1})$$

where $v(x, t)$ is the transverse displacement of the cable centerline, $x \in [0, l]$ is a spatial coordinate running over the chord of the element and t is the time.

By introducing the characteristic frequency $\omega_0 = \sqrt{(T/ml^2)}$ and the non-dimensional bending stiffness $e = \sqrt{(EI/Tl^2)}$, Eq. (1) can be re-written in the non-dimensional form:

$$e^2\, \partial^4_x u - \partial^2_x u + \partial^2_\tau u = 0, \qquad (\mathbf{2})$$

where $x = x/l$, $\tau = w_0 t$ and $u(x, \tau) = v(x(x), t(\tau))/l$. Values of e typical of stay cables are lower than 1%-2% [1, 8]. The small number multiplying the highest derivative hints the existence of boundary layers [3] and possible numerical difficulties if an appropriate rescaling is not used. General solutions of Eq. (2) can be expressed as $u(x, \tau) = f(x) \sin(l\tau - q)$, where l is a non-dimensional vibration frequency, q is a phase angle depending on initial conditions and $f(x)$ is a mode shape function. The vibration frequencies l and shape functions $f(x)$ are the eigensolutions of a fourth order boundary value problem defined by the ordinary differential equation:

$$e^2 f^{IV}(x) - f^{II}(x) - l^2 f(x) = 0, \qquad (\mathbf{3})$$

along with suitable boundary conditions modeling the cable restraints.

Ideal boundary conditions are often introduced, in the form of either perfectly hinged or perfectly clamped cable end sections, to simplify the analytical treatment of the problem (see e.g. [1, 4, 7-9]). As a first step towards a quantitative assessment of the effect of unknown boundary conditions on the modal properties of stay cables, a rotational spring is herein assumed to be attached to the cable end section at $x = 0$ (see Fig. 1). The degree of fixity of the rotational restraint is then defined by introducing the non-dimensional parameter: $r = K / (K + eTl)$, where K is the stiffness of the rotational spring. The parameter r takes values in the range: $0 \leq r < 1$, being strictly equal to zero for $K = 0$, i.e. for a perfectly hinged end section. The limit case $r = 1$, instead, is approached as $K \rightarrow \infty$, i.e. for a perfectly clamped end section.

By accounting for the definition of r, the boundary conditions for Eq. (3) read:

$$f(0) = 0,\; f(1) = 0,\; e^2 f^{II}(0) - (er\,/\,(1-r)) f^{I}(0) = 0,\; f^{II}(1) = 0. \; (\mathbf{4})$$

The general solution of Eq. (3) can be expressed as:

$$f(x; l) = A_1 \sin(z_1(l)\, x) + A_2 \cos(z_1(l)\, x) +$$

$$A_3 \exp(-z_2(l)\, x) + A_4 \exp(-\,(1-z_2(l))\, x)\,,$$

where A_i (i=1,…,4) are integration constants, while z_1 and z_2 are defined as:
(**5**)
$e\sqrt{2}\, z_j(\boldsymbol{l}) = \sqrt{[(-1)^j+\sqrt{(1+4e^2\boldsymbol{l}^2)}]}$, j=1,2. (**6**)
Substitution of Eqs. (5) and (6) in (4) yields the algebraic eigenvalue problem:
$\mathbf{B}(\boldsymbol{l};\ e,\ r)\,\mathbf{a} = \mathbf{0}$, (**7**)
where **a** is a column matrix collecting the integration constant A_i (i=1,…,4) and **B** is a 4×4 matrix whose entries depend on both the non-dimensional bending stiffness e and the degree of fixity parameter r.
A standard perturbation technique (see e.g. [6]) has been adopted in the present work to evaluate the eigenvalues $\boldsymbol{l}_k = \boldsymbol{l}_k(e,\ r)$ (k = 1, 2,…) of problem (7) for small values of e, which are typical of stay cables. Simple derivations, herein omitted for
the sake of conciseness, allow to obtain the following second-order accurate asymptotic expression:
$l_k\,/\,(kp) = 1 + re + [(kp)^2/2 + r^2\,]\, e^2 + O(e^3)$, $k = 1,2,\dots$. (**8**)
Once the eigenvalues $\boldsymbol{l}_k(e,\ r)$ are known, multiplication by the characteristic frequency $w_0(T,m,l)$ gives the natural frequencies of the cable: $w_k = w_0(T,m,l)\ \boldsymbol{l}_k(e,\ r)$, k = 1,2,… The closed-form Eq. (8) can be efficiently used within a frequency-based parameter identification strategy, whenever a set of measured frequencies is available from experiments. Standard vibration testing techniques (see e.g. [5]) can be applied to obtain the first M natural frequencies of a stay cable: w_1^*, w_2^*, …, w_M^* (with $M\geq 1$). The difference between calculated and measured natural frequencies, then, can be quantitatively assessed by introducing the cost function:
$F = S_{k=1,\dots,M}\,[(w_k^* - w_k(l,\ m,\ w_0\ ,e,\ r))^2/w_k^{*2}]$.(**9**)
By assuming that the length (l) and the mass per unit of length (m) of the cable are known without uncertainties, the unknown model parameters $\mathbf{p}=(w_0\ ,e,\ r)^T\in P\subset\mathbb{R}^3$ can be identified by solving the optimization problem:
Find $\mathbf{p}_{opt}$ such that: $F(\mathbf{p}_{opt})=\inf\{F(\mathbf{p})\}$, (**10**)
where the function $F(\mathbf{p})$ is calculated through Eqs. (8)-(9). Once parameters (w_0 ,e, r) are known from the solution of (10), the cable axial force can be easily calculated as $T=ml^2w^{\,2}$.
In the present work, the optimization problem in Eq. (10) has been solved through a custom implementation of the well-known Differential Evolution (DE) algorithm
[10] in the MATLAB environment. DE is a heuristic gradient-free direct search algorithm for optimization over continuous parameter spaces that utilizes NP parameter vectors as a population at each iteration. The initial population is randomly chosen and offsprings are generated by perturbing trial solutions with scaled differences of randomly selected population elements.

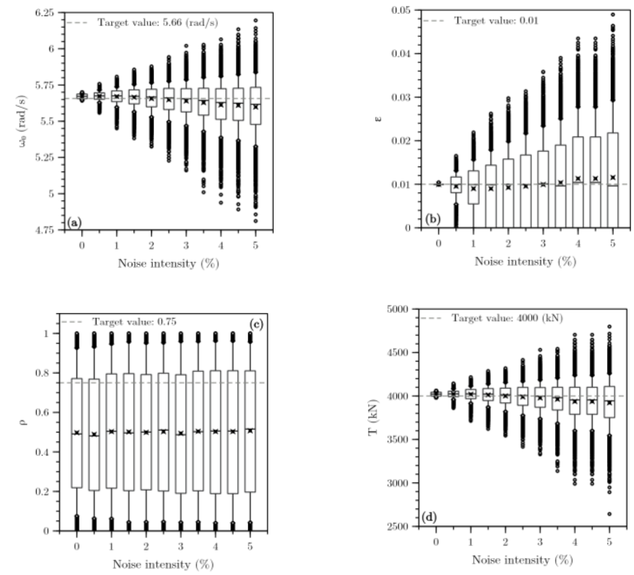
Application example

The performances of the proposed parameter identification strategy have been assessed through extensive numerical testing. Results will be presented in the following for a typical stay cable characterized by: w_0=5.66 rad/s, e=0.01, r=0.75, T=4000 kN, EI=1000 kNm^2.
In order to simulate experimental input data, the eigenvalue problem (7) has been numerically solved to get the first five natural frequencies of the cable. These frequencies, then, have been corrupted by multiplying the nominal values by a unit mean and low intensity Gaussian noise, to account for the effects of measurement errors. Different values of noise intensity, ranging from 0 to 5%, have been considered. For each noise intensity value, a sample of 5000 sets of noisy natural frequencies has been randomly generated. The optimization problem (10) has been solved for each set of numerically generated input natural frequencies by running the DE algorithm, starting from a population of NP=60 trial solutions randomly chosen in

the parameter space $P\subset\mathbb{R}^3$:$\{(w_0, e, r)$: $0\leq w_0\leq 1000$, $0\leq e\leq 1$, $0\leq r\leq 1\}$.
Figure 2 shows the results of the identification procedure as a function of the noise intensity. Statistics of the identified parameters w0, e, r and T are shown through box plots with whiskers corresponding to the 9th and 91st percentiles. Circles and stars denote, respectively, outlier points and mean values. The identification strategy gives fairly accurate results in terms of parameters w0 and e for all values of noise intensity herein considered, but is not able to correctly identify the degree of fixity parameter r. More in details, the identification procedure leads, for each value of noise intensity, to a mean value of r equal to about 0.5. This latter value coincides with the mean value of r within the randomly generated population members of the DE algorithm. The degree of fixity r, hence, doesn’t affect significantly the natural frequencies of the stay cable and cannot be accurately obtained through a frequency-based identification strategy. Direct inspection of Eq. (8) further supports the latter conclusion. Finally, it’s worth noting that, in spite of an imprecise identification of the degree of fixity of the cable end sections, the proposed procedure gives a very accurate estimate of the axial force,

Fig. 2. Variability of identified model parameters as a function of noise intensity.

with average errors always lower than about 2.5%.



Conclusion

An analytical model for transverse vibrations of a shallow cable with small bending stiffness and partially restrained end conditions has been presented. A closed-form asymptotic expression for the natural frequencies of the cable has been developed and used as the basis to develop a numerical parameter identification strategy. Numerical applications have shown that in spite of a highly imprecise identification of the degree of restraint of the cable end sections, the proposed procedure gives accurate and reliable estimates of the stay cable axial force.

Acknowledgments.
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Structural Investigations for the Refurbishment Project of the Municipal Building of Gjakova

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Abstract. This paper presents the structural investigations performed on the existing municipal building of Gjakova, intended to be refurbished as part of the Refurbishment project by the European Commission. The municipal building of Gjakova built in the end of 1960s is a reinforced concrete structure with bearing masonry walls which are scattered without any logical pattern. This structural system presents a difficult approach for the seismic assessment of this building. To obtain the confidence factors and the mechanical parameters for the appropriate numerical modeling, a series of structural investigations is carried out in detail. Initially a visual inspection is conducted, and a photographic survey is carried out with a detailed marking of each picture to visualize the building and plan the detailed inspection. The detailed plan is organized according to the gathered information and the existing documentation, so that maximal information about the structure is acquired with the minimal possible intrusiveness. The detailed inspection is accomplished in a series of frequent site visits and through the help of the measurement techniques a precise geometry is formed which supports the following phases. To obtain the correct mechanical parameters with very few damages to the structural elements a combination of destructive and non- destructive methods is utilized. Reinforced concrete core samples and reinforcement samples are drilled from the structural elements and combined with the rebound hammer measurements, a correlation is formed between these two. The reinforcement is located with the help of the profoscope and a discrepancy between the old drawings and the actual structure is noted. Afterwards, a spread-out testing utilizing the rebound hammer and the profoscope is carried out to achieve the required number of measurements. A compilation of all this information is utilized to assess the structural integrity of the structure and obtain the mechanical parameters. A qualitative measurement is performed for the structural arrangement of the elements and the regularity parameters are assessed as per the Eurocode 8 demands. With the data obtained from the measurements and the testing, reliable mechanical parameters and confidence factors are achieved to form a numerical model which represents appropriately the real building.

Keywords: structural investigation, NDT, refurbishment, retrofitting, RC structures

Introduction

The municipal building of Gjakova was built around 1960s with the purpose to serve as a municipal building of the city. There is no record of any change of use for the building and the new intended use of the building will remain the same. The total area of the site is 2344 m² and consists of the municipal building and the parking area. The existing five-story building consists of a basement, ground floor, three upper floors and an attic roof (Fig. 1). The existing space is not sufficient to accommodate the entire departments in one location and therefore a necessity for increase in space was essential. The solution was to remove the timber roof and accommodate another concrete floor and on top laying a green roof and making room for an open assembly hall. Furthermore, building a new annex building to the existing one. The scope of this study concerns mainly the existing building and thus the new annex building is of no interest. Since the structure is about 60 years old and is constructed in a period with different or no construction norms and especially with no seismic provisions, a careful and detailed assessment of the existing structure is done regarding its seismic capacity, structural stability and integrity. The structure is a public institution and is of high importance and therefore to have a more precise assessment a more thorough testing is to be done to achieve acceptable knowledge levels and confidence factors. This includes a preliminary inspection to be followed by a more detailed inspection in order to come up with acceptable structural parameters for the study. As such this study is concerned and is narrowed to the structural investigations performed on the existing structure of the municipal building of Gjakova.

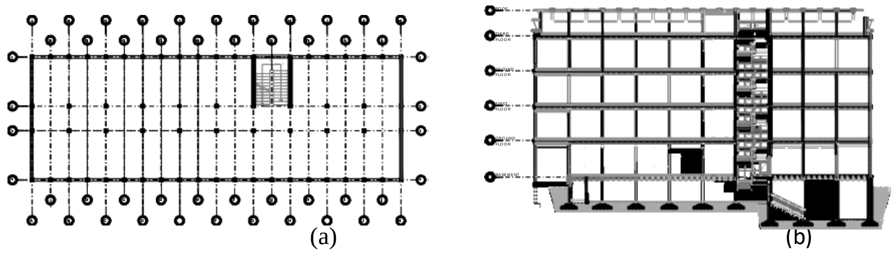


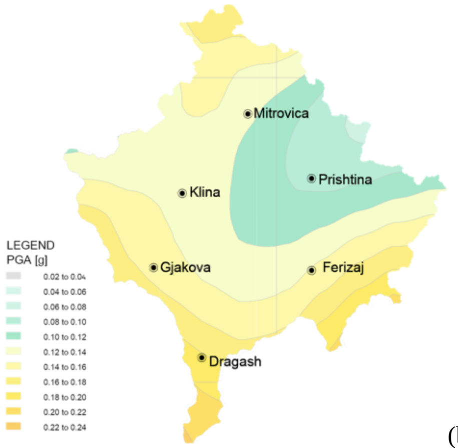
Fig. 1. (a) Existing base plan (Source: ARTING drawings); (b) Section drawing (Source: EPTISA drawings);

Seismicity of the Region

Kosovo does not have a national construction standard and thus no national accepted Seismic Hazard Map. Therefore, seismic analysis of structures is done either from previous maps, experience of engineers or with a careful and detailed assessment of the available resources and reports to come up with acceptable seismic parameters, which is done in the case of this study. According to [1], during 1456-2014 in Kosovo have occurred 152 earthquakes with magnitudes ranging from 3.5-6.3 in Richter scale. The ones relevant to the region of Gjakova are shown in Table 1 below.

Table 1. Relevant earthquakes to the region of Gjakova that occurred during 1456-2014

Region	Prizren	Peja	Gjakova	Prizren	Klina
Date	16 Jun 1456	11 Nov 1662	03 Sep 1922	26 Sep 1945	05 Feb 1947
Magnitude	MS = 6.0	MW = 6.0	MW = 5.3	MW = 5.0	MW = 5.2
Intensity	8.0	8.0	7.5	7.0	8.0



(b)

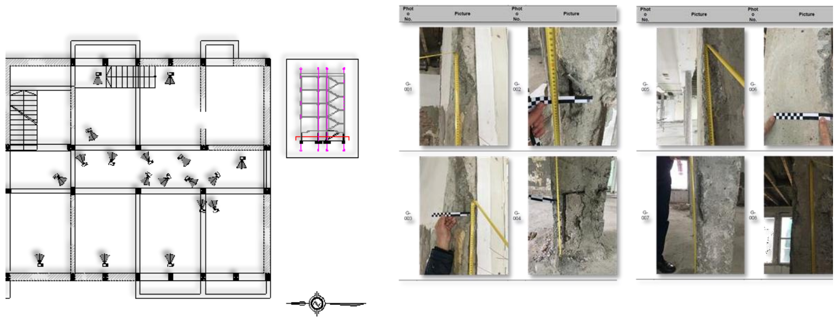
(c)

Fig. 2. (a) Seismic hazard map from [1]; (b) Seismic zonation from the former Yugoslavian construction standard; (c) Seismic hazard map from [2];

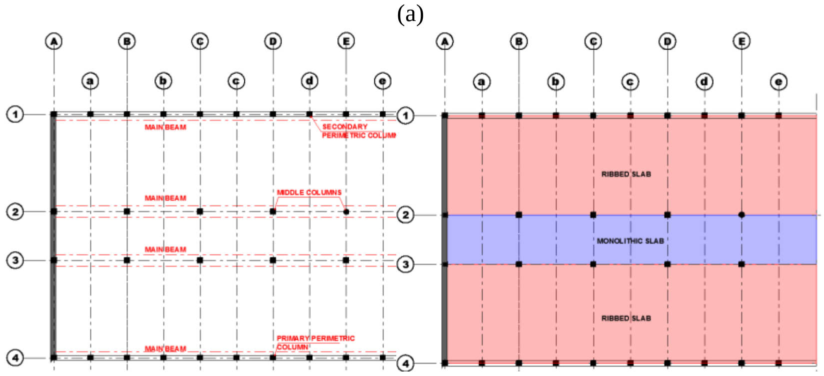
The aforementioned study also produced a seismic hazard map for Kosovo with PGA values for 10% of exceedance in 50 years with a return period of 475 years (Fig. 2-a). Furthermore, the map of Yugoslavian Construction Standard of 1964 (Fig. 2-b) presents a seismic zonation in the MCS scale which positions Gjakova in zone VII and correlates to PGA values between 0.18g-0.34g. Another study [2], extended their seismic research in Kosovo producing a seismic hazard map for 10% probability of exceedance in 50 years with a return period of 475 years (Fig. 2-c). Concluding all this information the PGA values for the region of Gjakova range from 0.10g-0.34g, and the most suitable value to be used further in the analysis is 0.20g.

Preliminary Inspection

A preliminary inspection was carried out in the structure to perform a qualitative investigation in order to obtain information about the structural system, the actual damages and decide on the testing to be performed on the structure. Initially, original drawings from the archive were analyzed and then processed and regenerated into CAD drawings. A detailed geometrical survey was required to verify the existing structure to the original drawings. New and more detailed drawings of the existing situation were created with the geometrical in-situ data obtained. The geometrical survey was complemented with a photographic survey which was also mapped in the drawings (Fig. 3-a). The photographic survey included the documentation of the different damages in the structure. The damages were additionally gathered in a table where their nature and severity were categorized to have a better understanding. A major number of serious damages were found in the structure including structural



(a)



(b)

Fig. 3. (a) Example of the photographic survey and mapping on the drawings; (b) Structural system of the existing building;

cracks, spalling and pop-outs of the concrete, extensive segregation, corrosion and exposed rebars, misalignments, warping, rebar buckling and installations going through the structural members and so on, where some can be seen in Fig. 3 (a).

The structural system of the building can be seen in Fig. 3 (b), where the vertical bearing system consists of reinforced concrete (RC) columns in the middle and perimeter whereas on the lateral sides and for the stairs core structural masonry is observed, thus creating a mixed structure. The horizontal bearing system is made of a combination of monolithic RC slabs in the mid-span and RC ribbed slabs on the sides, all supported in RC beams which go only in the longitudinal direction of the building. The foundations consist of RC isolated footings for the mid-columns and RC strip foundations for the perimeter columns and walls with no tie beams to connect them.

Detailed Inspection and Structural Investigation

The detailed inspection included structural investigations in the structure utilizing the non-destructive testing (NDT) and destructive testing (DT). The main focus was to assess the in-situ compressive strength of the concrete conform [3] and to quantify the actual condition of the structural members. The method of obtaining the concrete strength includes coring of multiple concrete samples, but due to the fragility of the structure only 4 cores were taken, namely 3 columns and 1 beam. The core sampling procedure shown in Fig. 4 followed the localization of the rebars with a profoscope (1,2), surface preparation with abrasive stone (3), 9 rebound hammer readings in the coring location (4), fixation of the core drilling machine (5) and then the coring of a 10 cm diameter concrete sample with no reinforcement inside (6).

(1) (2) (3) (4) (5) (6)

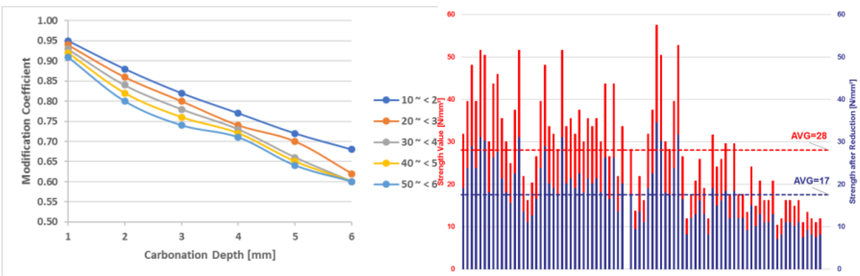
Fig. 4. Core sampling procedure;

Afterwards, a widespread NDT was carried out in the entire structure conform [4] taking 9 readings with the rebound hammer in each location avoiding the rebars which were initially located and measured with the profoscope. The NDT testing was performed on 52 columns (x9 readings) and 33 beams (x9 readings). Additionally, 6 steel samples were taken to be tested in tension, namely 3Ø10 mm and 3Ø12 mm.

The concrete cores and the steel samples were tested via destructive methods in a local laboratory conform [5,6] and the result processing is shown the following section. The concrete cores were additionally tested for carbonation depth by applying the phenolphthalein, conform [7].

Information Processing

The concrete compressive strength in-situ conform [3] can be obtained by correlating the NDT results with results from DT of cores. Initially the NDT results from the rebound hammer readings are reduced depending on the carbonation results of the cores. The average carbonation depth in the samples was found to be 27.5 mm, which is a relatively high value. The reduction values (Fig. 5-a) are given by a modification coefficient which multiplies the rebound hammer reading depending on the detected strength in MPa. It can be seen that the theoretical value for 27.5 of carbonation reaches a modification coefficient of zero, but for practical reasons readings are done for 6 mm of carbonation. The rebound hammer readings are shown in Fig. 5 (b), where can be observed the reduction of the average compressive strength from 28 N/mm² to 17 N/mm².



(a) (b)

Fig. 5. (a) Carbonation reduction factor according to strength detected; (b) Strength readings from rebound hammer and after reduction;

Following the procedure in [3], the correlation values are obtained by initially forming a basic curve and then shifting it as per the test results from the in-situ cores and the rebound readings. The final curve obtained, which correlates the rebound hammer readings to the in-situ compressive strength, is shown in orange in Fig. 6.

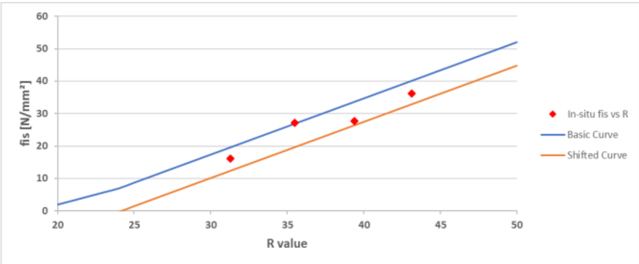
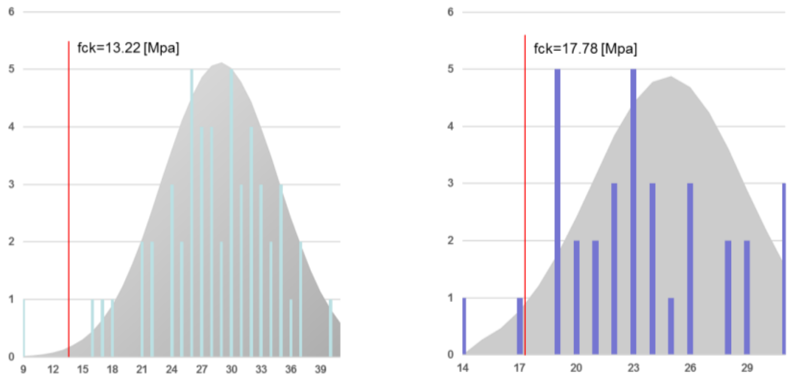


Fig. 6. The curve correlating rebound readings to the in-situ compressive strength;

The characteristic compressive strength of the concrete, which is directly related to the concrete class, is then obtained from the expressions available in the standard and the correlated values from the graph in Fig. 6. The characteristic compressive strengths are found to be 13.22 N/mm² and 17.78 N/mm² for the columns and the beams, respectively. A normal distribution of the compressive strengths for the columns and the beams which resulted from the graph above is shown in Fig. 7. It can be easily observed that the characteristic compressive strength values are at about 5% area that defines the characteristic compressive strength.



(a) (b)

Fig. 7. Normal distribution of the compressive strengths for (a) columns and (b) beams;

From these relations it can be concluded that the concrete class for columns can be set as C12/15 and for the beams C16/20, which are way lower than the classes foreseen in the initial original design. The results for the steel samples are shown below in Table 2 and it can be seen that the class can be set to the original steel class of the design which was S 240/360 and be on the safer side.

Table 2. Steel sample results

Sample	Diameter [mm]	Yield Strength [MPa]		Ultimate Strength [MPa]		Elastic Modulus [MPa]	
	Ø	fy	mean	fu	mean	E	mean

10-1	9.90	355.2		378.2		58527	
10-2	9.80	362.1	324.3	396.8	394.0	95263	125652
10-3	9.80	255.6		407.0		223166	
12-1	12.10	417.3		477.1		865999	
12-2	12.10	426.3	404.7	474.3	477.9	190318	411733
12-3	12.10	370.4		482.4		178881	

The material values to be used for the assessment of existing structures should be reduced with confidence factors according to the knowledge level of the structure as per the guidelines set by [8]. A knowledge level KL2 is obtained with the information available for the structure which results in a confidence factor of $CF_{KL2} = 1.20$.

Conclusions

The extensive studies carried out in the municipal building of Gjakova has provided with a better understanding of the structural state of the current building.
From the initial inspection it was able to achieve a thorough observation of the damages and deformations in the structure and achieve the desired level of geometrical accuracy. This helped in the next phase of analysis which included the numerical simulation of the entire building to assess its structural stability when subjected to external loading, especially seismic action. The structural investigation via non-destructive and destructive testing provided with the most valuable information on the structure which is directly applicable in the numerical model and provides a high level of accuracy in the model. The overall information aided to the decisions which were made in the later stages where retrofit or demolition were a matter of questioning.

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A Geospatial Analysis of the Existing Flood Situation in the Buna River Basin

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Abstract. Flooding events cause economic, social and environmental damage and lives loss. In Albania, the rivers constitute the highest flood risk. Basic knowledge for apprehending the flood risk concerns the frequency and intensity of floods, the exposition of humans and assets to flooding, their sensitivity to floodwater and their susceptibility to suffer damage. My study case is located in Buna River area. This is a zone with frequent flood risk. A detailed analysis of the current state, problems and issues of the area is structured around topic areas elaborated in detail. The assessments, maps and developed catalogues of measures shall fulfil the obligations of the "European Directive 2007/60/EC on the assessment and management of flood risks". According to the Directive flood risk management shall address all aspects of floods risk management, including prevention, protection and preparedness.

Keywords: Flooding1, Buna River2, Flood Risk Menagement3, European Directive4

Introduction

Flood risk and vulnerability are increasing due to changes in rainfall pattern, increased frequency of extreme events, changes in land use and development in flood prone areas as a result of socio-economic demand. Human lives, property, environment, and socioeconomics are at increasing risk due to flooding [1]. The vulnerability is a multi-layered and multidimensional social space defined by the determinate, political, economic and institutional capabilities of people in specific places at specific times [2]. The natural dynamic evolution of a river and the adjacent morphological environment are particularly important for the communities that concentrate in these areas their socio-economic activities. In addition the anthropogenic pressure plays a fundamental role in influencing the rate of the fluvial processes. The combination of both factors involves all the major rivers, so that today it is not easy to find long fluvial reaches free of human intervention [3]. Integrating spatial planning and flood risk management is a sustainable approach to deal with flood issue [4]. Spatial planning influences the critical factors of flood risk such as location of activities, type of land use, scale of development and design of physical structures [5]. Spatial planning can influence those critical factors on different spatial scales, from local plans to national or even international strategic plans [6]. The Buna basin located in the North Western Albanian Region of Shkodër is prone to severe floods which occurred regularly in the last years and might increase in frequency and intensity due to climatic changes in the region. The latest major floods in January 2010, December 2010 and March 2013 resulted in high economic and environmental losses. Without adequate adaptation to the increased flood risk, social, economic and health damage are likely to increase. One of the main objectives is the minimization of damages cause by floods in the coming years. In general climate change projections for the region show an increase in temperatures as well as an increase in frequency and intensity of floods [7]. While summer periods will be dryer, latest figures of the European Environmental Agency predict an increase of heavy precipitation in the winter period between 5-15 %. Especially climate variability will increase and more extreme events are likely to happen [8].

Object of the Study

1. Examine the flood situation in Albania and analyses the factors of river flooding in this area.
2. Identify European and Albania flood management approaches in planning and practices.
3. Analyze Albania present situation and suitable prospects on flooding in the study area.

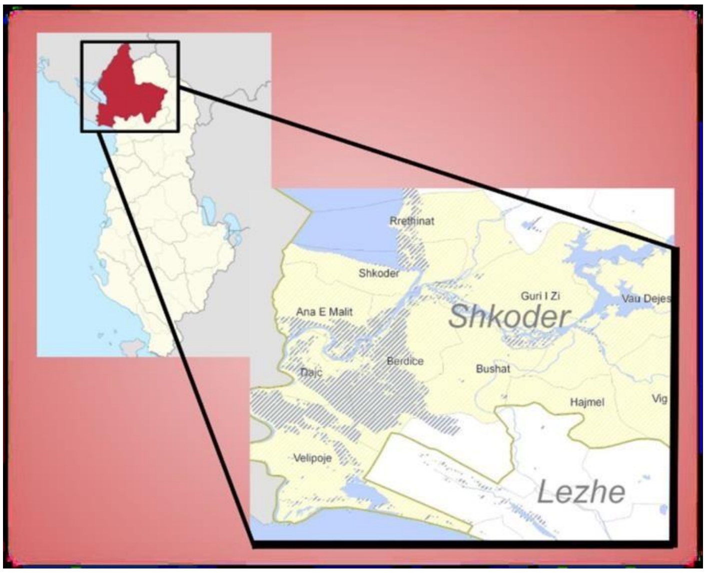

Figure 1: Highlighted in red is the Shkoder region of Albania. The detail shows the municipal units in Shkoder. [9]

Study area

The The River Buna runs in the last North-west segment of the Albanian- Montenegrin border. This river springs from Lake of Shkoder, quite close to the city of Shkoder, between the hill of "Rozafa" Castle and Taraboshi Mountain. Buna is the only emissary of the Shkoder Lake. First, Buna runs to the south, alongside Taraboshi, further it snakes toward west and then it takes again the south direction, up to its mouth in Adriatic Sea. The Albanian-Montenegrin border traverses the River Buna from village Samrisht in Albania and Gorica in Montenegro, continuing up to the river mouth. The river has a length of 44 km. Lake Shkodra, Drin and Kir Rivers, drain into the Buna, which flows towards and empties into the Adriatic Sea.

Figure 2: Drin and Buna River Basin – overview. [10]

The combined flow from the other rivers and the lake into the Buna River can sometimes add up to more water volume than the Buna River can hold, causing it to overflow. Floods are frequent during the November-March period, when the region receives about 80-85 percent of its annual precipitation [11]. This potential risk area in the Shkodër region covers the communes Ana e Malit, Bërdicë, Bushat, Dajç, Gur i Zi, Rrethina, Shkodër and Velipojë,

Figure 3: Satellite image of the Drin and Buna rivers and the northwestern flood plain: 1) Komani dam, 2) Vau-Deja dam, 3) Shkodra town, 4) Shkodra Lake, 5) Drin River, 6) Buna River. [12]

Factors that Contribute to Flood Risk

Climate Change

In general climate change projections for the region show an increase in temperatures as well as an increase in frequency and intensity of floods [7]. There are two climate scenarios that we have looked at called Representative Concentration Pathways (RCP). The first, less extreme scenario is RCP4.5, where greenhouse gas (GHG) emissions peak near 2040 then decline. The second, more extreme scenario is the RCP8.5 or “business-as-usual” scenario, where GHG concentration continues to increase by 2100 [13]. Average annual precipitation has already decreased up to 20% in the region compared to the base period of 1961-1980. While the RCP4.5 scenario predicts no significant change, the RCP8.5 scenario expects annual precipitation to decrease by another 20% in the years 2046-2065 [13]. However, seasonal precipitation is changing, with higher levels in December, January, and February followed by drought periods in June, July, and August throughout the Western Balkans and the Shkodër region, as can be seen in Figure 4 [14].

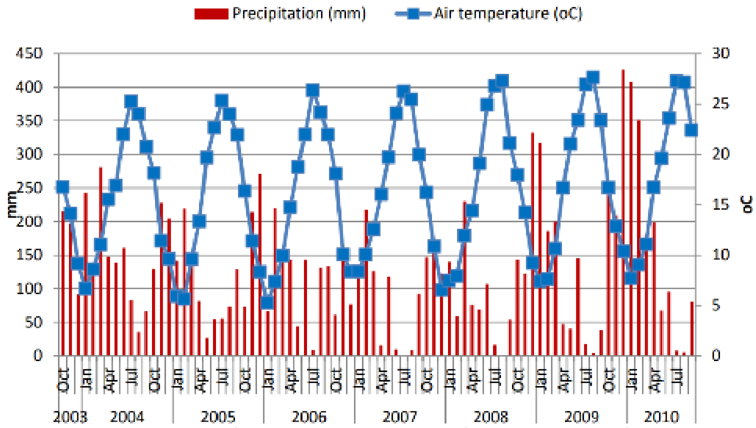

Figure 4: Monthly average rainfall and air temperature for the stations of Ulcinj, Bushat, Dajç and Velipoje in the period 2003-2010 showing the increase in peak rainfall for the fall and winter months [14].

Erosion

Erosion increases the severity of flooding from the Buna River. In some areas, erosion is changing the shape of the river, making it wider but shallower, increasing the probability of overflow and flooding in surrounding farmland. Erosion is a natural effect that can be worsened by anthropogenic interventions such as dredging and deforestation. For example, in 2015, severe floods in southern Albania were attributed to river erosion caused by deforestation of the river margins where the river meets the land [15].The extent of erosion can be seen in Shkodër both in the Buna River (Figure 5). In addition to exacerbating the effects of floods, erosion along the Buna River deteriorates the integrity of the soil layer at river margins by washing both the nutrient-rich topsoil and its base away [9].

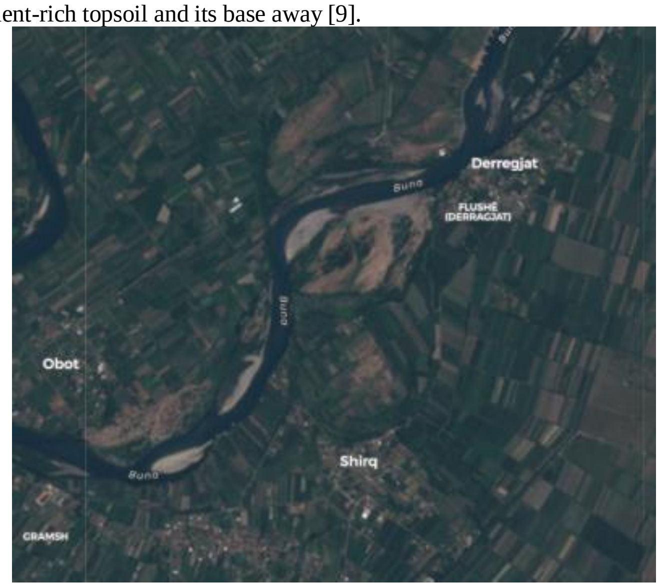

Figure 5: Satellite image from 07-21-2018 showing erosion (in grey) along the Buna river south of the city of Shkodra acquired through Modified Copernicus Sentinel data [2018]/Sentinel Hub [16].

Hydropower Plants and Dam Management

The construction of major hydropower plants and dams in Shkodër since the 1970s has also contributed to flood risk in the area. The three major hydropower plants in the Drin River are the Fierza (1978), Komani (1985), and Vau I Dejes (1971) dams with their respective power plants (Kesh, 2017). The Fierza dam, is located closest to the source of the Drin River and the Kosovo border. From there, the Drin flows to the Vau I Dejes dam where the river basin begins to empty to the sea. Shkodra is located slightly northwest of the Vau I Dejes dam. These three hydropower plants produce 95% of Albania's power, and excess energy provided by these dams is sold to neighboring countries [17]. The series of dams can help reduce the area flooded in severe events as they block river residues, especially gravel, and hold large amounts of water upstream. Conversely, mismanagement of the dams can increase flood risk. After analyzing heavy floods in 2010, experts concluded that dam operators of Vau I Dejes delayed opening the spill gates until the reservoirs were full and structural damage was imminent. This required operators to release a peak of approximately 3,500 cubic meters per second, more than seven times above the normal release of 500 cubic meters per second [18].

Figure 6: Drainage channels converging south of Dajc

Figure 7: Gravel pile from dredging by the Buna Rive

Hydrogeology

Significant part (~24%) of the area consists of karst limestone formations, which offer considerable potential for groundwater exploitation. The coastal aquifers interact with the sea, including in the form of submarine groundwater discharges, which contribute to the creation of brackish water habitats in the coastal zone. A significant portion (~54%) of the plan area is classified as having low and very low vulnerability to groundwater pollution. Only 7.2% of the area can be considered as having very high and high vulnerability to groundwater pollution. [14].

Table 1. Main hydrogeological formations and their extent in the plan area [14].

Formations	% of total
Fissured and karstified porosity aquifer with high to very high permeability	23.56
Fissured porosity aquifer with medium to low fracture permeability	0.45
Moderately productive intergrannular aquifers with medium to low permeability	29.83
Practically impermeable rocks without considerable intergranular or fissured porosity	25.67
Productive intergrannular aquifers with very high to medium permeability	20.49

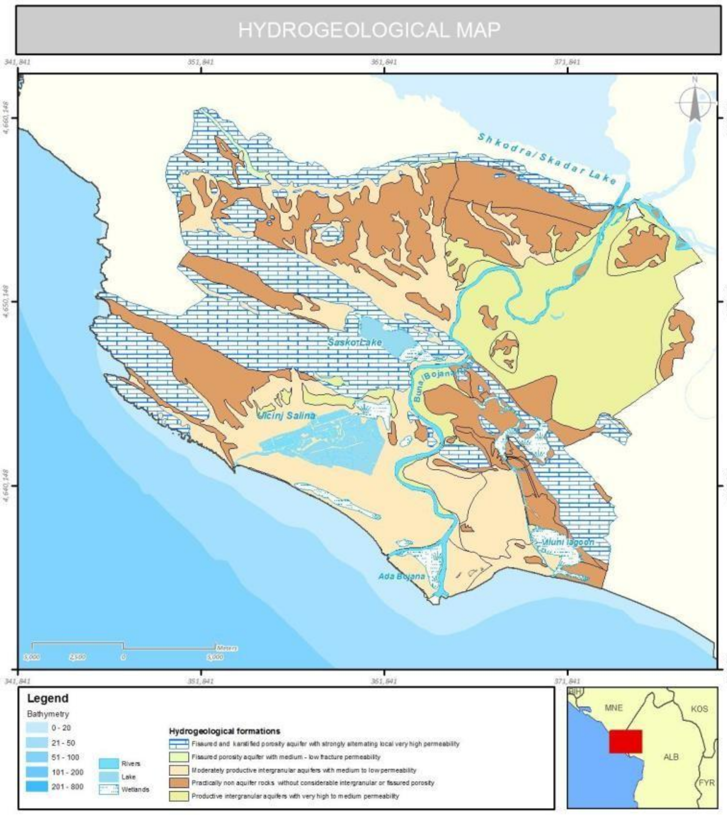

Figure 8: Hydrogeological map of the Buna/Bojana basin [14].

Sea level rise

The sea level rise has a special significance because it highly contributes to factors that cause flooding, The Albanian Costal area is prone to subsidence that might intensify the impact of sea level. By averaging the results for Albanian coastal area results that sea-level is likely to increase averagely up to 40cm, reaching the maximum level of 73 cm by 2100 [19].

Table 2. Projection of changes in annual sea level (cm) [20].

	Sea level rise (cm)	
years	2050	2100
aver	14.6	37.8
max	26.4	72.6
min	7.2	15.2

Figure 9 shows the flooded area due to maximum projected sea level rise. Combined effect of sea level rise and storm surges may cause even more extensive flooding in the area.

Figure 9. Sea Level Rise in Plan area in Albania [20].

Buna river present situation in the study area.

The existing flood protection system dates from the 1960ies and relies on a series of dikesand drainage channels [21]. These have been constructed mainly after the catastrophic inundation of the years 1962-63. In this time huge investments have been carried out in the lower part of the rivers Drin and Buna.

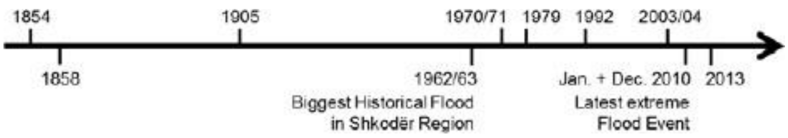

Figure 10: Historic mayor flood events since 1851 [21].Based on available data – no further mayor floods documented since 1851

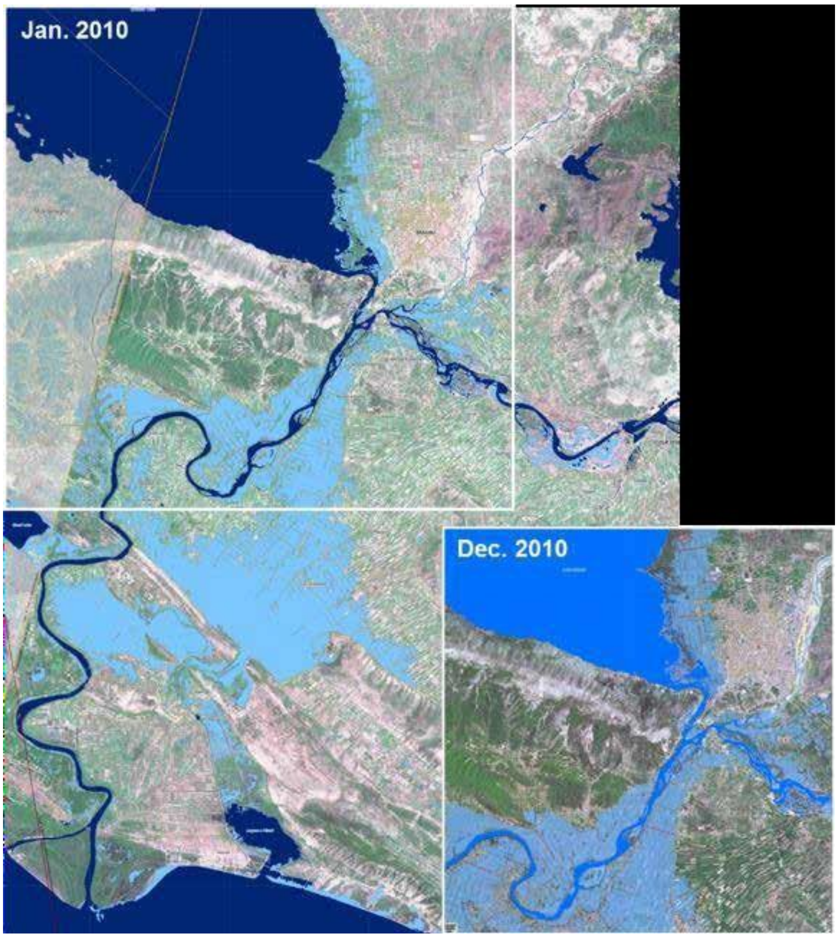

Figure 11: Satellite Image of flood extent in January 2010 and December 2010 (TerraSAR and Radarsat - DLR, 2010) **[24]**

Figure 12: Flood of December 2010 (NATO, 2010)

Figure 13: Farmland near Shkodra in November 2018 (Left) and flooded in December 2010 (Right)

European Flood Directive a new approach for study of fluids

The "European Directive 2007/60/EC on the assessment and management of flood risks" contributes to setting a legal framework for integrated water management including flood risk management for all European member states. It builds up on the change of strategy in fighting against flood risks: the traditional approach was to protect people, economic goods and agricultural land from floods (which regularly fails when extreme floods overtop the protection works). The modern approach of the directive is to cooperate with all relevant actors to "life with the floods", to protect if possible, to adapt uses and constructions to flood risks in respective areas and especially to prepare for being flooded in a holistic approach with all potentially affected people, organizations administrations and businesses [25].

Figure 14: The FRM cycle according to the EU FRM Directive **[26]**

Management cycle, from preparation and disaster management to the recovery phase. Development and agreement on the regional and local FRM plans, including a common picture of the flood risk management measures and further activities for the region and the communes. [27]

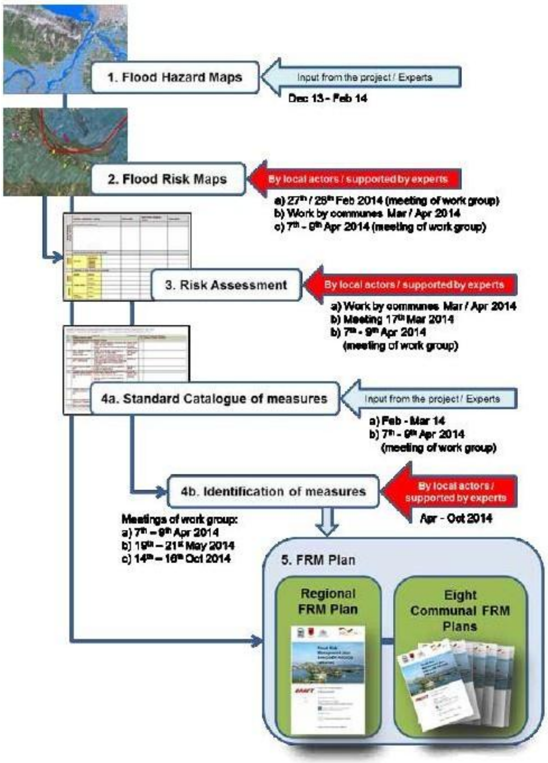

Figure 15: Working steps for the FRM plan **[27]**

Conclusion

The frequency of natural hazards, such as floods, have been increasing during the last decades. On the other hand modern societies have become more vulnerable to impacts of flooding. Necessary measurements for flood protection are: Topographic Field survey of flooding area, Remote Sensing comparison of satellite images between years, Measurement of hydrological data through years. Determination of high-risk areas and flood damage caused by floods of different sizes. According to the EU floods directive which have to be implemented in all member states it is necessary to do the identification of potential significant flood risk areas and to create Flood hazard and flood risk mapping. The assessment and identification of areas of potential significant flood risk should include the existing data, local knowledge and GIS-analysis for the preselected areas.

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BIM-based risk identification and assessment in building projects at their design phase

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Abstract. The complexity and dynamics of construction sites, in combination with the uncertain behaviour of human factors, result in considerable workplace injury, illness, and fatality statistics in the construction industry despite the strict legislative framework worldwide. Therefore, the demand for a thorough risk management process, based on automated safety modelling and preventive strategy, stands out in order to identify and eliminate potential hazards early in the design phase of a construction project, resolving thus safety issues in the field by extending traditional safety management practices. The objective of this study is to investigate whether and how Building Information Modelling (BIM) can be used within the health and safety framework to enhance risk identification and assessment in building projects at their design phase. To this aim, a case study is conducted via a BIM software, namely Revit, to develop an effective Building Information Model of a two-storey building in which safety measures are introduced according to State legislation and field practices at a specific construction phase. In the proposed way, the real-life complexity of the risk management process is simplified, due to the object-oriented approach of BIM, the variety of BIM libraries and the experiential recognition of unsafe conditions with 3D simulations in place of the non-judgmental and merely bureaucratic law-enforcement methods. In conclusion, BIM enhances the communication between engineers and workers, using interactive tools, and facilitates the Safety Officer duties in the direction of preventing potentials hazards from the early planning phases.

Keywords: Building Information Modelling (BIM), construction safety, health and safety plan, prevention through design, risk Identification, risk assessment, risk matrix

Introduction

According to the Occupational Health and Safety (OHS) statistics, workplace injury, illness, and fatality percentages in construction industry remain globally high. More than one third of all US workplace fatalities occur in the building construction, while the Finnish construction industry is responsible for one out of four fatal occupational accidents [1].

As these statistics indicate, the demand for a thorough risk management process, based on automated safety modelling and proactive strategy, stands out in order to identify and eliminate potential hazards early in the design phase of a construction project. The process will resolve thus safety issues in the field by extending traditional safety management practices.

Building Information Modelling (BIM)

Nowadays, the use of Building Information Modelling (BIM) has been developed rapidly in the Architecture, Engineering and Construction (AEC) industry due to the collaboration and communication enhancement that offers among all project partners. The method increases productivity and quality, while reducing project cost and delivery time [2]. BIM and BIM-related technologies are considered useful tools to overcome the existing obstacles in traditional risk management methods. For instance, BIM itself has been proven as an effective way to assist early identification and assessment of risks for design and construction through 3D visualisation [3], 4D scheduling [4], and 5D cost estimating [5]. With the growing establishment of BIM in the AEC industry, some attempts that could further integrate BIM with risk management have been conducted, e.g. automatic rule checking [6, 7, 8], proactive IT (Information Technology)-based safety systems [9], and safety training in a virtual gaming environment [10].

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Risk Matrix

The Risk Matrix is a tool to estimate levels of all identified risks of a project through a two-dimensional diagram. The diagram axes represent the likelihood of occurrence and the risk impact respectively [11]. This color gradation chart is often uased during a Risk Assessment to determine how likely a risk is to manifest, and how severe the outcome could be if the risk occurs. There are various versions available. In this research the one appeared on Table 1 is chosen.

Table 1. The Risk Matrix (source: E. Mantle, D. Story & A. Rees, 2015)

	Impact	1	2	3	4	5
	Probability	Negligible	Minor	Moderate	Significant	Severe
5	(81 – 100) %	5	10	15	20	25
4	(61 – 80) %	4	8	12	16	20
3	(41 – 60) %	3	6	9	12	15
2	(21 – 40) %	2	4	6	8	10
1	(1 – 20) %	1	2	3	4	5

The overall risk rating is determined by multiplying the rating given to the impact by the rating predicted for the probability, using the qualitative scales (1 - 100% or 1 - 5 = Very Low to Very High or Negligible to Severe).

RISK = Hazard Probability x Hazard Severity [12] **(1)**

Its objective is to quantify the risk in order to assist decision making management by setting priorities and determining the action required to control the hazards [13], as shown in Table 2 [14].

Table 2. Sample guidance on action required by different risk levels (source: Tony Boyle, "Health and Safety: Risk Management", 4th Edition, 2018)

RISK RATING	**ACTION REQUIRED**	
1	I	• No action

2 - 6	II	• Record hazard • Record risk ratings of hazardous events • Recommend upgrades to risk control measures if a risk rating is not as low as reasonably practicable
8 - 16	III	• Record hazard • Record risk ratings of hazardous events • Record the recommended risk control measure needed to reduce the risk ratings to below 7 or as low as reasonably practicable
20	IV	• Seek further advice
25	V	• Stop the activity immediately • Seek further advice

Methodology

The followed methodology for the proactive strategy via Building Information Modelling (BIM) from the design phase is based on literature review and online research. According to the outcomes, a process was developed and applied in a Case Study.
The process is summarized in the following steps:

- First, the Building Information Model of the construction site of a two-storey typical building project is developed through the Autodesk® Revit software. The model contains architectural and structural information. All protection measurements required by the State legislative framework and described in the Health and Safety Plan have been incorporated in the model.
- Subsequently, ten (10) frequently identified at construction sites scenarios of unsafe conditions are simulated. In each of them, risk identification and assessment are carried out according to the risk matrix.
- Finally, risk reduction measures are proposed per scenario.

Case Study

In the framework of the Case Study, a two-storey building project is considered at its construction phase. Specifically, it is assumed that the foundation, the building frame, the slab placement at ground and first floor, the roof and the infill walls of ground floor have been in place. In addition, a hole for the upcoming stairway placement is supposed to be present at the current phase as well as an open excavation with the foundation and the perimetric walls of the residence cesspool. The chosen geometry can be characterised simplified in order to facilitate the research objectives.

The developed Building Information Model, depicted in Figure 1, represents the ideal construction site of the building as far as the safety standards. In the model, BIM libraries were used extensively to facilitate the process and reduce the effort.

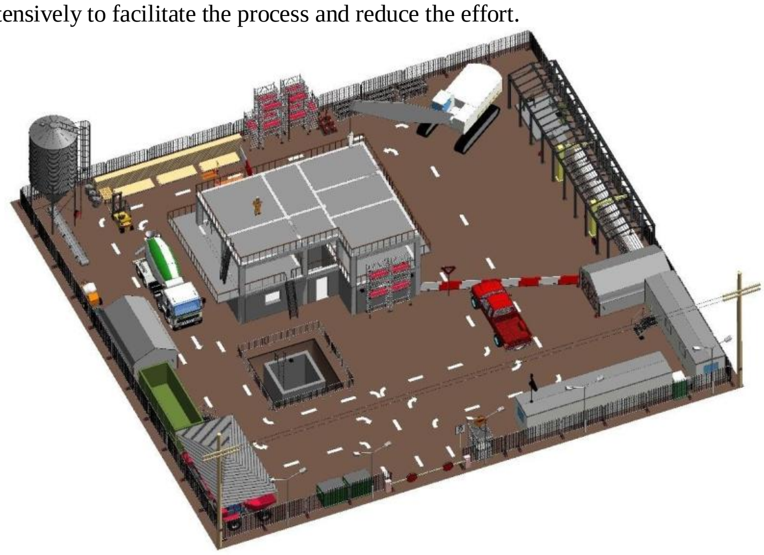

Fig. 1. 3D view of the construction site with the applied safety measures in a BIM form

The construction site follows the typical lay-out of a building project and comprises the following facilities, depicted in Figure 2:

1. Two-storey building
2. Cesspool well
3. Temporary protection railing around cesspool well
4. Vehicle entry
5. Entrance guard room
6. Worker entry
7. Workplace office
8. W.C.
9. Locker rooms
10. Closed warehouse
11. Cutting and shaping reinforcement bar facility
12. Exterior fencing of construction site
13. Lifting machine
14. Outdoor scaffolding & shuttering storeroom
15. Temporary plastic jersey barriers around lifting machine arm orbit
16. Forklift
17. Cement silos
18. Concrete production unit
19. Covered aggregate warehouse
20. Debris bin
21. Covered parking, maintenance and repair facilities
22. Trash bins.

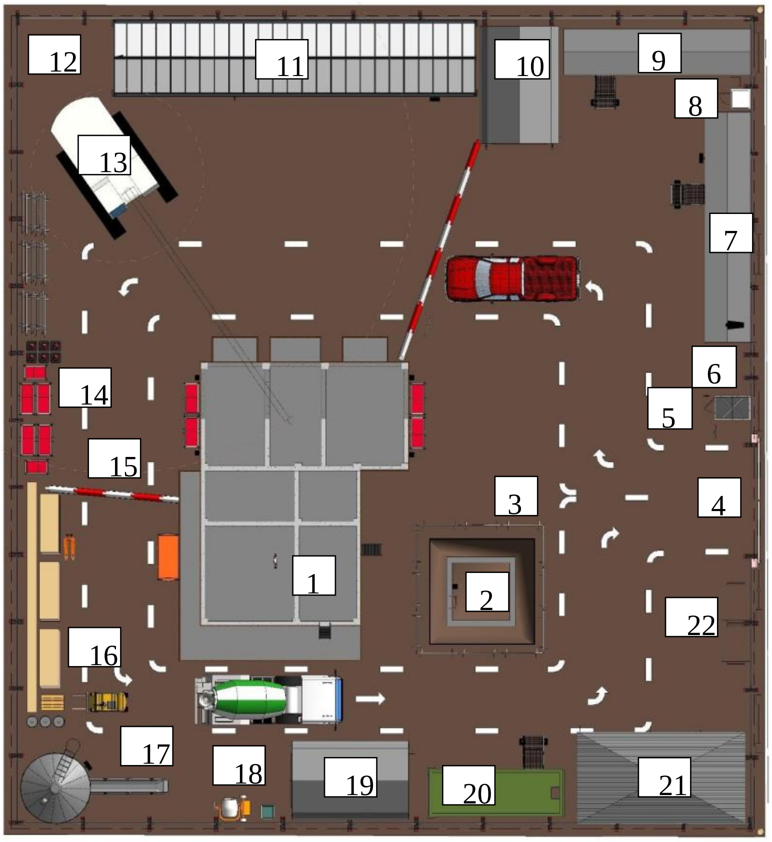

Fig. 2. Construction site facility arrangement with safety measures in BIM form

The simulated scenarios of unsafe conditions are the following:

1. Removal of temporary protective railing around the cesspool well.
2. Removal of exterior fencing allowing citizen entrance in the construction site.
3. Removal of temporary plastic jerseys/forklift and worker exposure to the lifting machine arm orbit.
4. Absence of temporary and durable railing against fall around the roof slab.
5. Absence of temporary and durable railing against fall around the hole of 1st floor slab.
6. Absence of durable handrail at 1m, parallel board between floor - handrail and shields on either side of scaffoldings.
7. Not stabilized portable staircase, at base and top, without handrail at 1m in its entire length.
8. Existing disorder at the outdoor scaffolding & shuttering storeroom.
9. Lack of signs.
10. Impingement of the lifting machine arm to the aerial electricity network.

In the following section, results are indicatively presented for the 4th scenario, depicted in Figure 3, which rather represents one of the most frequent unsafe condition at construction sites.

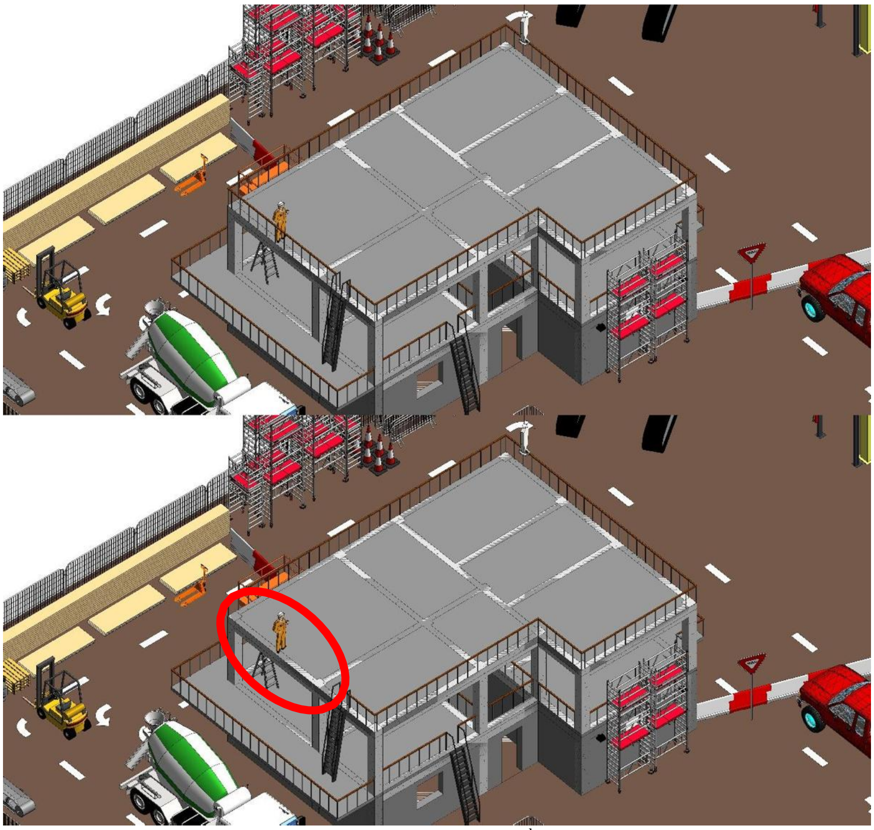

Fig. 3. Representation of the unsafe conditions for the 4th scenario: Absence of temporary and durable railing against fall around the roof slab

The BIM representation facilitates the process of visually identifying and, more importantly, assess the characteristics of the unsafe conditions. In this example, the probability of occurrence may be considered as medium (41 – 60% or level 3), following the fact that protective measures generally exist in the construction process, but the main reason for hazard manifestation is the non- durable railing against fall, due to their quick and rough placement. The impact of the fall is considered to be severe (level 5), as a likely fall from this height may result in severe or fatal injury. Based on these considerations, the total risk parameter takes the value of $3{\times}5 = 15$. This value places the risk in the orange section of Table 1 while, according to Table 2, the actions required are of type III and the proposed measure for the risk reduction is the proper perimetrical placement of temporary and durable railing against fall of height at least 1m, with handrail, intermediate bar and shield on slab ends.

The same procedure is followed for all unsafe condition scenarios described above. The risk assessment and the proposed actions for the individual scenarios are depicted in Table 3.

Table 3. Risk assessment and proposed action types for the case study scenarios

No.	**Scenario of Unsafe Conditions**	**Level**	**Hazard Type**	**Hazard Probability**	**Hazard Severity**	**Risk**	**Action Type**

1	Removal of temporary protective railing around the cesspool well	Site	Fall	4	(61-80) %	4	Significant	16	III
2	Removal of exterior fencing allowing citizen entrance in the construction site	Site	Bury	2	(21-40) %	5	Severe	10	III
3	Removal of temporary plastic jerseys/forklift and worker exposure to the lifting machine arm orbit	Site	Bury	5	(81-100) %	5	Severe	**25**	V
4	Absence of temporary and durable railing against fall around the roof slab	Roof	Fall	3	(41-60) %	5	Severe	15	III
5	Absence of temporary and durable railing against fall around the hole of 1st floor slab	1st Floor	Fall	4	(61-80) %	5	Severe	20	IV
6	Absence of durable handrail at 1m, parallel board between floor - handrail and shields on either side of scaffoldings	Site	Fall	5	(81-100) %	4	Significant	20	IV

7	Not stabilized portable staircase, at base and top, without handrail at 1m in its entire length	1st Floor	Slip	4	(61-80) %	4	Significant	16	III
8	Existing disorder at the outdoor scaffolding & shuttering storeroom	Site	Ergonomic	4	(61-80) %	3	Moderate	12	III
9	Lack of signs	Site	Crash	2	(21-40) %	5	Severe	10	III
10	Impingement of the lifting machine arm to the aerial electricity network	Site	Electrical	3	(41-60) %	5	Severe	15	III

Conclusion

The paper presents the role that BIM can play within the risk identification and assessment in building projects during their design phase, encouraging further dissemination and application of the method in construction practice and upgrading the safety procedures compared to the fragmentary and partial approaches that typically take place nowadays. In this direction, a BIM model of a two-storey building project is presented, in which safety measures are introduced according to State legislation and field practices at a specific construction phase. Following, ten common hazard scenarios are developed and their risk parameters and proposed remedy actions are demonstrated.

In the proposed way, the real-life complexity of the risk management process is simplified, due to the object-oriented approach of BIM, the variety of BIM libraries, and the experiential recognition of unsafe conditions with 3D simulations in place of the non-judgmental and merely bureaucratic law-enforcement methods. Moreover, BIM enhances the communication among engineers and workers, using interactive tools, and facilitates the Safety Officer duties in the direction of preventing potentials hazards from the early planning phases.

In future research, a continuous real-time monitoring of the construction site will be tested in order to identify unsafe conditions that deviate from the BIM model, advise on safety measures, and promptly inform the Safety Officer.

Nomenclature

AEC (Architecture, Engineering and Construction) BIM (Building Information Modelling)
IT (Information Technology)

OHS (Occupational Health and Safety)

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Review of pollution sources of Sitnica river, Kosovo and approaches for improving

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[1] Wastewater treatment,
Prishtina, Kosovo

Abstract. The paper makes a review the point sources, pollution of Sitnica River in the section from its springs to discharge into Ibër River. All the existing emitters discharging into the river and its tributaries are analyzed and ranked according to their impact on the river body in terms of pollution load. The main source of pollution appears to be the untreated domestic and industrial wastewater, as the pressure increases with the increasing of population living in the river basin.
Different wastewater management approaches (centralized and decentralized) are proposed for the most significant emitters in order to mitigate the anthropogenic impact on the river water quality.

Introduction

Water is natural source, which is used from people to develop their life, with water we secure our food and other basic elements of life. Every activity in our life, some less some more are connected to water .Without water there is no life. With development of urban areas, there are higher request for drinking water and in the same time there will be request for the treatment of wastewater.
Therefor water should be used carefully, with professional administration. With development of human civilsation, there is necessity for administration of water sources, especially so deficit won't show up at the dry seasons when we need water mostly.
By increasing the measures and standards for water quality protection, requirements for wastewater treatment before being released to the recipient will be increase significantly
Stormwater is another topic that belongs to management of wastewater, this system, which is necessary to protect nature, people, and infrastructure from damages.
Use of water or water resources should be under fundamental regulations, which are:
Water is irreplaceable substance for life. Without water there is no life. According to latest statistics 4/5 jobs depend on water (M.Brilly V. 2016), this shows the importance of water for people.
- At all stages of human development water has been a common (public) asset. Even Roman law defines common property "Flumina omnia publika sunt - Rivers are public property" Therefore, there are rare states that have privacy over rivers. Therefore, water resources should be guarded by the state administration, local administration and any individual.
- This rule indicates the democratic rules for the utilization and conservation of water resources. This is where the principle of sub solidarity lies, where decisions have to be proposed from the lowest possible level, such as local governments and municipal bodies.
Therefore, it is important to analyze the solution of the form of wastewater treatment which is economically viable and to achieve all the parameters of the wastewater treatment before pouring into the recipient. The operation of cleaners must be effective in both qualitative and

quantitative terms. For illustration, several forms of biological treatment of municipal water have been considered.
To analyze the forms of treatment plants, the wastewater discharge points as well as the amount of water must be identified.

General Description of Sitnica River Basin

The Sitnica River with source Sazlija pond ,north Ferizaj,runs along the plain of Kosovo, length of the river stinica is L=90km and avarage Discharge Q=4.3m3/s, which forms the central, northern and eastern parts of Kosovo. In the western part of this area the river is surrounded by the mountains of Qiqavica, Golesh and Cerraleva. In the eastern part it borders with the Prugovc, Kozenica and Zhegovc Mountains in the north it borders with the Kopaonik mountains**.**Sitnica River Basin includes municipalities: Mitrovica, Vushtri, Podujevo, Obiliq, Fushe Kosove, Pristina, Lypjan and Shtime, Drenas and part of Ferizaj. The Sitnica River contains some small river branch discharges, that have small amounts of water, except for the rivers Llapi, Drenica and Shtimjanka. These three rivers have significant amounts of water. The exception is the Shtimjanka River which brings large amounts of water in the fall, winter and especially in the spring when there is snow melting. During this season there is flooding of agricultural land and settlements.

Watershed area

The catchment area is F = 2 848km2, the direct catchment area is Fdr = 418km2, with discharges: on the right side Fd = 1953km2 and on the left side Fm = 895km2. the catchment covers about 50% of its surface. This includes the Drenica River catchment area F = 433.4 km2, the Gracanka River F = 97.0 km2.

Therefore the amount of water coming from these rivers has to do with the part that has been regulated. This includes only the Shtmlanka, Gadimka, Gushterica and Qellopek, Gracanka and Prishtevka rivers, which in certain cases bring significant amounts of water.

Avarage hight of the river is.

$$H_{mes} = \frac{546.50 + 490.80}{2} = 519.00 m.l.m.$$

Sureface of watershed with plants and tree s

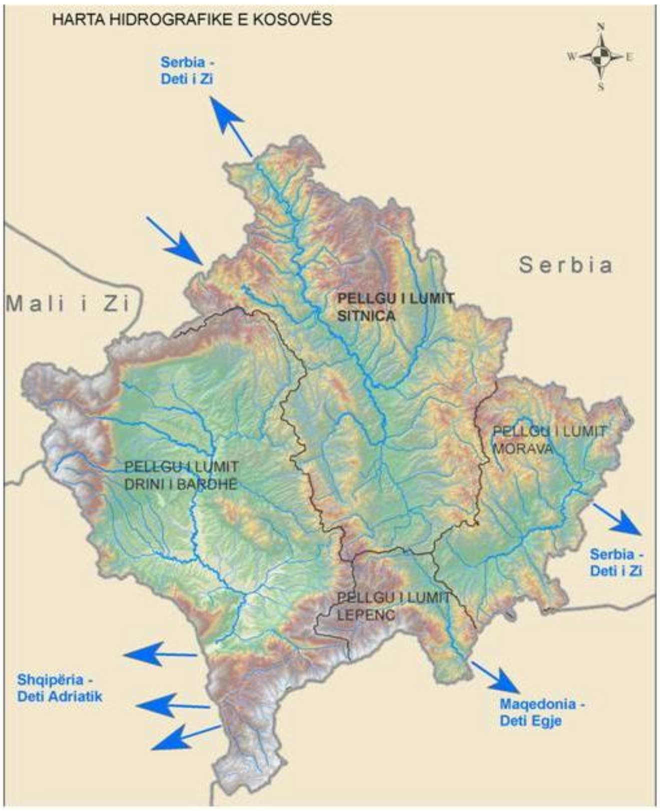
$$\gamma_m = \frac{F_{mal}}{F_n} \cdot 100 = \frac{1245}{2848} \cdot 100 = 44\%$$

Hydrology

Hydrological data and their processing have been used by the "Kosovo Hydroeconomic Basis" with additions. These are old datas, since former Yugolsavia, there are no systematically new datas.

Avarage flow perrenial is 1923 – 74 s.m.Nedakoc				
Flow modul V ($m^3/s/km^2$)	Koeficienti i rrjedhj	Flow average $Q_{mes}(m^3/s)$	flow max./mes. $Q_{max}(m^3/s)$	flowmin. $Q_{min/mes}(m^3/s)$
0.00259	0.143	2.024	23.76	0.0480

$Q_{/min}=0.18m^3/s$
$Q_{average}=4.30m^3/s$
$Q_{ssmax}=55.01m^3/s$
$Q_{max}^{25}=415.50\ m^3/s$



Separation of water flows in Kosovo

In high rainy seasons such as autumn, winter, and spring, the Sitnica River floods many agricultural areas, roads, settlements, and economic objects, as full waters cannot swallow the natural riverbed. During the summer the Sitnica River has small amounts of water and sewage discharge into it causes high pollution in it.These means there are no working or management of this river only pollution added

Most of the settlements are supplied by drinking water supply systems. Also, many settlements have sewage networks for the disposal of waste water. These waters are released untreated into recipients. Especially the settlements with large number of inhabitants where the amount of wastewater is large and their discharge into the recipient causes high pollution.

River pollution rate

Therefore, given that this river passes through many agricultural areas and settlements, it is of particular importance to protect it from pollution. Water pollution of this river is of great importance for the health of the country's population. Who live next to him. Near the riverbed there are agricultural lands in which many different crops are cultivated. Water pollution in the river has a great impact on the environment, agriculture etcT

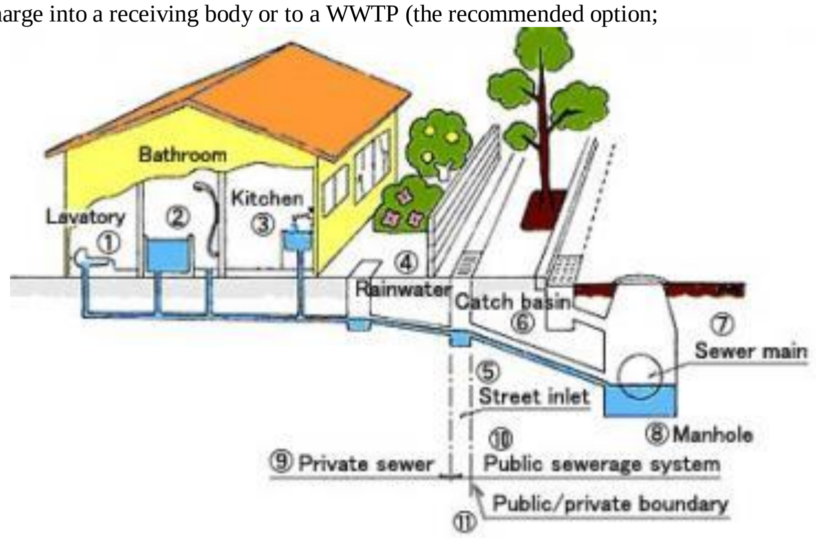
Discharge of domestic wastewater into the river Shtimjanka

Pollution of river Prishtevka from wastewater

Wastewater

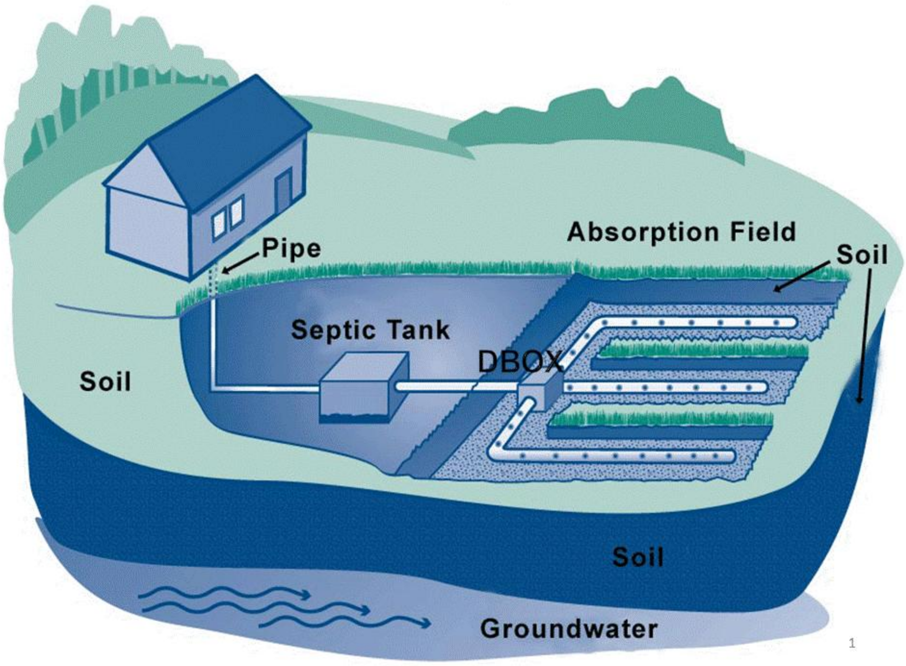
Types of wastewater management

Centralized – the water is conveyed from the place of origin through sewer collectors, which discharge into a receiving body or to a WWTP (the recommended option;



http://archive.sswm.info/print/1573?tid=710

Decentralized – the water is collected (treated) at the place of origin or very close to it (i.e. the so called individual systems)



http://www.masslocalinstitute.info/wastewater/Wastewater4.html

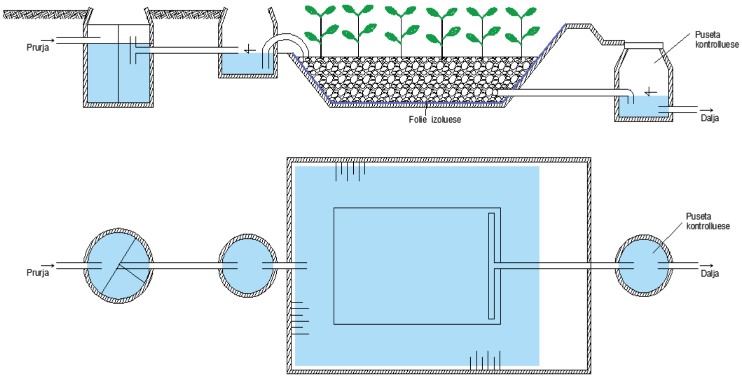
Biological wastewater treatment

Since over 60% of the population live in rural areas, the creation of wastewater from these sites can be purified in plants that are effective and financially accessible. Since many objects need to be constructed numerically and as such they must have low construction and maintenance costs.

So far there are several such facilities built and have proved to be quite effective where the satisfactory rate of wastewater treatment has been achieved. These settlements have a population of up to 5,000.

For settlements with a population of up to 10 000 inhabitants, wastewater treatment plants of a biological type are used. This form of purification of sewage from small settlements significantly reduces the degree of water pollution in the river.

When designing these facilities, consideration must be given to calculating the number of occupants accurately and the surface area required for the installation of water purification facilities. Care must also be taken in the design of the filtering surface.



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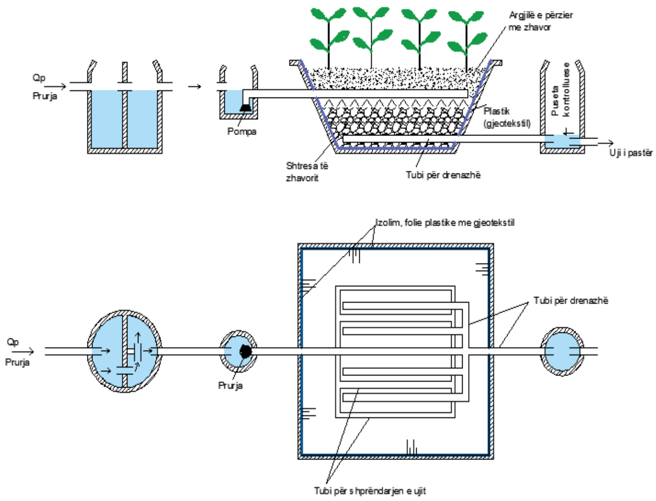

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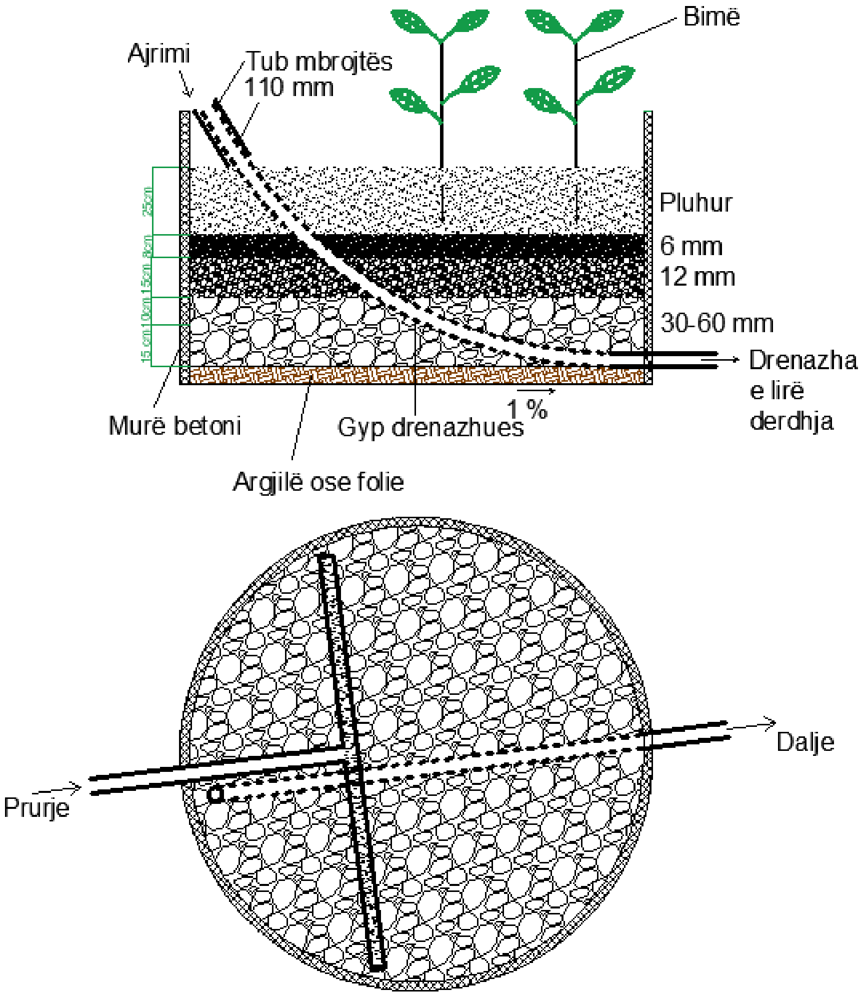

Conclusion

The River basin includes municipalities: Mitrovica 84.235 inhabitnants, Vushtri 26.964inhabitants, Podujevo (23.453 inhabitnats), Obiliq (21.549 inhabitants), Fushe Kosove (33.977 inhabitnats), Pristina (204.721 inhabitnats), Lypjan 57.605 inhabitants and Shtime (27.324 inhabitants), Drenas (58.531 inhabitants) and part of Ferizaj (108.610 inhabitants). The total population in the river basin is 562.818..and presents 31% of the population of Kosovo

This paper analyzes and studies the simplest and fastest potential that can yield results for the Sitnica River pollution, considering the large Sitnica River Basin, which is the second river in Kosovo, with large settlements and rural, it turns out that in order to achieve a rapid effect on

river pollution, with low cost facilities and not taking much time for construction and activation - which are presented in the paper is one of the proposals that should be used for protecting rivers from further pollution - otherwise facilities that treat large settlements require financial, technical and political support to achieve wastewater treatment where appropriate. Therefore, consideration of the construction of small river clearing facilities in rural areas is effective and expeditious.
The superscript numeral used to refer to a footnote appears in the text either directl
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MORE SPACE IN AIRSPACE CAPACITY

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This work describes, explains and proves necessity for stabile global air track network World Air Track Elastic Network – WATEN to accommodate future air traffic and enable full automation in process of control and separation of civil aircraft during en route phase of flight under instrument flight rules - IFR.

According to long term forecasts global air traffic will rise in average of 5% per year threatening to overload the airspace and limit future development of civil air transport. New solutions for improving the safety of the flight by automation and increasing airspace capacity through more efficient airspace use will enable continuous increase of number of flights.

More then 70% of incidents and accidents in civil aviation are caused by human factor therefore a strong need exists for higher level of automation in civil air transport especially in process of control and separation of civil aircraft that are flying under instrument flight rules. Automated systems will separate traffic by using appropriate equipment, software and procedures. The first phase will exclude air traffic controllers' and their active control of traffic and in the next phase pilots will be excluded too.
One of basic conditions for full automation in process of control and separation of air traffic is setting of permanent global network of air tracks named: World Air Track Elastic Network – WATEN as a physical base for aircraft systematic movement and matrix for separation procedures. It will be established as permanent but with physical and commercial elasticity and based on direct tracks that are part of earth's great circles connecting the most important destinations in the World as a Strait Line Network. Other destinations will be included in Secondary Network. Aircraft using this network, Global Navigation Systems and WATEN MODE equipment on board of aircraft will have possibility to separate themselves from the other aircraft in same or opposite direction of flight as well as to cross the track of other aircraft in horizontal or vertical plane without external help. Lack of airspace capacity will additionally stimulate a need for completely new approach to aircraft separation. Group flight, which is now reserved for other types of flight, will become legal part of regular procedure for commercial aircraft. Automatically separated traffic will enable for higher level of safety, reduction of number of occurrences, greater capacity of the airspace and more stabile flow of civil air traffic.

Keywords: Air Track., Network, Group flight, Capacity

Introduction

Airspace capacity relevant to this work is greatly dependent on such airspace organization conditioned by needs and influences of the airspace users.
According to the users' demands, on macro plan, the airspace organization shall be done in a way to satisfy, to the greatest extent possible, users' global needs and in the same time adjusting

the portions of airspace to the local needs of the Member States. The needs to be satisfied are generally set to the possibilities to finance ATM[(1)] system of a Member State or group of Member States, if the function is united at functional block, FABA[(2)] level or joint services and to assist the financing of regulatory and supervisory function of the Member States.
The initial losses of capacity are occurred during the global planning due to the fact that all portions of the airspace are without homogenized regulations, technical equipment and procedure development, so the capacity of a portion of the global network may be expressed through the capacity of its weakest part. This is the reason for homogenizing of the airspace characteristics of the specified elements.
In certain States the developed regulation has enabled certain parts of the commercial aviation to be relaxed from unnecessary costs by applying certain savings in respect of manpower and equipment according to the law. Examples for this may be found in more intensive use of electronic self-serving of the passengers (computer reservations system, e–booking, remote e–banking, e–check–in etc.).

Euro control technical development

Technical development of control system and monitoring of the air transport have cut the costs in respect of manpower. Modern communication systems and aviation supervision have enabled the possibility to organize the civil and military air traffic controls and monitoring service in centers covering more and more territories. Sophisticated regional centers so called functional airspace blocks will follow and control air transport in some joined Member States only from one center thus ceasing the need for organization of smaller area control centers in each country, and in some more than one center there is only a tendency to unite in future air traffic controls of all European air transport from one center EUROCONTROL [(3)].

High level of procedure development enables the traffic operations, at less important airports, to carry out and without presence of air traffic control authority. Such procedures are based on principles of airspace classification according to the criterion on various needs for different types of air services operation, from which two of them are basic, that is, visual flight rules and instrument flight rules; for carrying out the supervision/control of the air services operation according o the needs of users. Sophistication of aviation (approach and departure, as well as missed approach) procedures has achieved such high level enabling the commercial flights operated with commercial transport aircraft for carriage at medium distance to be carried out at airports without air traffic controls.

Member States are with various social system/order, great differences in respect of the state administration organization and efficiency, differences regarding the aviation practices and infrastructure development which directly affect on the differences of capacity of the respective airspaces. Negative influence on capacitance results from the elements such as territorial aspect of certain Member State (the ratio of a dimension of state territory or a portion thereof may be 10 : 1 or worse as the case is with Chile), in which a part of airway network has been arranged to its character aiming to set up the most frequent destination along its longest dimensions thus enabling a successful commercial effect and at the same

[(1)] ATM – Air traffic Management

[(2)] FABA- Functional airspace block in approach

[(3)] EUROCONTROL – The European Organization for the Safety of Air Navigation time reducing the unit rate (unit rate is a right of every country to establish, through the cooperation with the neighboring countries and international organization and aviation industry associations on the base of a unique mathematical coefficient, the base for calculation of air traffic services charges on the territory and airspace for which the respective air traffic control is in charge). Due to the various methodologies used for calculation of coefficients and differences in applying stimulating and disincentive tariffs regarding the services charge are occurred which in addition to the influence on the scope and frequency of the air services within the respective country, that is, within the part thereof, determine the commercial efficiency of the neighboring countries even of physically detached countries which consecutively due to the deviation of the traffic by the change of the charge collection structure in certain countries may increase the served traffic or decrease it, and in extreme cases may dense the airspace or block the air traffic flow.

Political and economical factor

In certain cases the political and commercial factors are contradicted in certain neighboring members (there are examples of such contradictions of interests even in members which are not neighboring, but the consequences of intervention on network portions may set them on contrary positions because such interests of different members are complementary, that is, there is no possibility to accomplish all of them in the scope, but they form constant value and they may only share such constant value in various ratios. Direction network which would be set only according to the global indicators criteria would create different political and commercial effects to various members. Such a different would be defended by common global interest reconciling respective interests unlike local network development which could not avoid local influence, and thus it may not be led to the common overall benefit criteria.
By comparison of possible and realized routes commercial losses caused by political reasons may be illustrated in Figure 1.

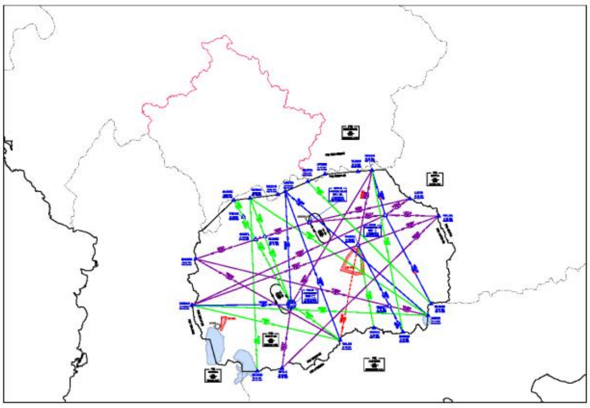

Fig.1 Image of Macedonia and Kosovo border with routes used by aircraft taking-off and landing at airport in Pristine.

A sufficient number of ports/connections at global network are left for local purposes, while the local created networks with described characteristics condition the ports/connections quality, so the difference in allocation of accessibility to global network and main traffic flows could not be compensated. (Global traffic flows are variable due to the influence of a great number of factors, so the position and influence of the members conditioned by the same global criteria will be variable).

For the purpose of conducting the efficient airspace organization and its capacity increase at global community level, it is necessary to establish common principles based on mathematical calculations representing commercial interests of the airspace users (even though they are owned by certain Member States or are part of their economic system, thus, to a certain extent, being representatives of the interest of that members), as well as objective criteria. (objectivity is a subjectivity conditioned by the participation intensity and possibility to influence on decision making process, which should be substituted by a consensual decision by the Member States instead of majority decisions. It could be possible by means of compensation of injured members, because the air transport would achieve an additional value enabling the compensation by increase of capacity and thus the conditions for commercial efficiency).

Increase airspace capacity

Meteorological influences on airspace capacity would be to the greatest extent possible compensated by globalization of network routes unlike the existing practice in which the traffic diverting has due to meteorological obstructions been carried out within the borders of one (each respective and often in contrary to or discrepancy to the diversion carrying out in the neighboring Member State, because there is no standardized procedure regarding the united diverting in the region, nor in certain countries with several area control centers, which are not coordinated in that sense) Member State or at most in cooperation between two Member States at the borderland, while the global network would have the possibility to avoid adverse meteorological conditions by turns which involve a great number of members, regions even continents.

Favorable meteorological conditions are a unit measure of meteorological influences, that is, they are a minimum of such influence enabling the maximum airspace capacity measured by means of such criteria, and this is called basic airspace capacity according to the meteorological influence criteria.

Fully agreed commercial influences are commercial influence unit rate enabling maximum airspace capacity measured by such criteria and it is called basic airspace capacity according to the commercial influence criteria.

Allocation of the airspace capacity allocation and management systems capacity and traffic control are territorially different. The most successful approach can be carried out by means of application of a global strategy. Capacity increase of airspace capacity as well as of operative systems capacity, at local level, within one member or group of members would be nullified by insufficient capacity of the neighboring member or group of members.

Airspace capacity decrease shall increase the loading of control system and air traffic management by increased activity of system resources, first of all, of human resources.

All above mentioned factors and influences make the airspace capacity to have dynamic changes. Dynamic changes shall have backward effect on the basic capacity.

Conventional networks of air markings decrease the total capacity due to the fact that the aircraft are laterally separated by minima routs separation, while in using of elastic marking, the lateral aircraft separation has been achieved by minimum aircraft separation from aircraft. Conventional networks achieve maximum capacity per unit of time thus preventing traffic flow in desired term. This imposed measures which decrease the network load by equal traffic allocation during 24 hours. The elastic networks may, by increased capacity, handle/receive traffic increase during peak periods/terms. In such a way, the commercial effect of the traffic users has been increased, both individual and total.

Having in mind the fact that the airspace capacity is a dynamic value, an available instrument for future capacity prediction is required. Total capacity at conventional networks increase, limited by capacities of network parts. Due to the fact that there is possibility for local influence on great number of the said factors, global control of such network, which satisfies the contradicted demands of local communities, is impossible to be carried out. By establishment of global network such global influence would be eliminated (if the weather influences are not seen as local factor, because they in their nature are not such) and the number of capacity limitation factors would be decreased, as well. Such smaller number of factors would be possible to control at a global level by a single network management system. Smaller number of factors would also be possible to predict, thus enabling a successful forecast of the future airspace capacity in a tactical, utility, strategic and development sense. Reducing the capacity decrease factors (we are not talking about increase of capacity, because a single capacity is in the same time theoretical maximum capacity that is undisturbed capacity. The task of future investigators is to show airspace capacity in a form of equation/formula which variables would be globally controlled capacity decrease factors).

Any activity in the field of airspace management is directly dependent on its capacity. In present state there is a diverse direction of influences that is the capacity depends on airspace management, as well. Such a state will be present until there is local influence on the global capacity, as described above. When such influence ceases to exist, final airspace organization and modeling will be possible such moving it to the maximum capacity according to this criterion that is according to the airspace organization and classification.

Automatic support

Technical requirement for such management is support of semiautomatic and automatic support systems.

Processing technique and software solutions have achieved the required speed and capacity level to support the operation of such systems. Due to the lack of prerequisites till now there has been impossible to set such aims till now. Control and air traffic management semi–automation has to the great extent increased the airspace capacity.

Further improvements may be expected by aircraft optimization and standardization. Present commercial fleet is composed of different aircraft types. It is worth mentioning that special effect on capacity has the difference in speed as well as the difference of the effect of turbulent air movement on aircraft on routes because suspension of reduced vertical separation minimum, RVSM[(4)] may be caused by which the airspace capacity is amounted to 2002 when this program

[(4)] RVSM - Reduced Vertical Separation Minimum was realized and when by its application the growth of number of flight levels by 6 was achieved thus improving the airspace capacity enabling the increase of flights number in regions where the program was realized and in 2011 during which a return to standard increase of over flights number in Europe of 3 – 4% per year according to the IATA[5] Report (International Aviation Transport Association – IATA), was noted. In addition, the effect of air mass turbulent movements is dangerous for airspace capacity due to the different weights of aircraft engaged in commercial air traffic by which application of method for aircraft separation in turbulence conditions per different categories (ICAO [6]Doc 4444, ATM Chapter – Separations) has been caused. Anticipated future work on optimization of commercial aircraft types would, due to the decrease of such risks, effect on increase of airspace capacity.

Eexperientially and use of commercial aircraft led to empirical knowledge (visual at products sold by the two biggest commercial aircraft factories in the world, Airbus and Boeing), that the future development of commercial air fleets will not be based on offers diversity trying to cover greater number of segments demand for aircraft types used at various routes and different load capacity, but by spreading of Hub and Spoke airport network organizations the needs of most airlines would be covered by two basic types for long and regional distances. Having in mind that those two basic aircraft types are not used within the same airspace portions (aircraft engaged in regional operations use lower altitudes unlike those connecting Hubs that is carriages on sub-continental and intercontinental routes) so a greater fleet percent is unified in such segmented airspace portions enabling the increase of capacity.

When total unified of two types covering the needs of Hub and Spoke transporting systems were achieved, the separation applying the unique separation minima and thus homogenized use of the airspace at volumetric unit enabling increase of airspace capacity would be allowed.

The subsequent phase would be the decrease of such separation minimum by applying of new safety models by which additional production standards resulting in additional homogenization of transport commercial aviation would be enforced**.**

Summary

Airspace capacity has been defined as capacity of a portion of the airspace handling/serving certain phases of aircraft operations, air transport as well as of military planes operations. Such capacity shall be calculated on the base of aircraft physical characteristics such as airspeed, size and weight and their mutual influence while using the common airspace by required application of vertical separation minima on the ground of above mentioned characteristics as well as legal restrictions and influence of adverse weather conditions.

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An application of PCA based k-means clustering for customer segmentation in one luxury goods company

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Abstract. In this paper K-means clustering algorithm is applied in order to classify customers into several groups showing the similarity within a group is better than among groups. After determining the relevant client's attributes in a SQL Server database, PCA (Principal Component Analysis) is applied in order to reduce the number of features, and after that, and K-means algorithm is performed in MATLAB programming environment, using fixed number of clusters. Each centroid defines one of the clusters, while each data point is assigned to the nearest centroid, based on the squared Euclidean distance. In this research, centroids are randomly generated, while the separation distance between the resulting clusters is analyzed and illustrated using the Silhouette index. The analysis and results presented in this paper could determine a similarity in purchasing or using the services by a population cluster in one luxury goods company, to develop market segments, to identify repetitive behavior or trends in order to evaluate client actions and to create some new customer loyalty campaigns.

Keywords: Cluster analysis, K-means, Principal Component Analysis (PCA), Silhouette index

Introduction

K-means clustering is one of the simplest unsupervised machine learning algorithms [1]. Typically, unsupervised algorithms make inferences from datasets using only input vectors without referring to known, or labeled, outcomes [2].
The objective of *K*-means is to group similar data points together and discover underlying patterns [3-5]. To achieve this objective, *K*-means looks for a fixed number of clusters in a dataset. *K*-means algorithm identifies *K* number of centroids, and then allocates every data point to the nearest cluster, while keeping the centroids as small as possible [6], [7]. To process the learning data, the *K*-means algorithm in data mining starts with a first group of randomly selected centroids, which are used as the beginning points for every cluster, and then performs iterative (repetitive) calculations to optimize the positions of the centroids [8-10].
In this paper *K*-means is applied in order to segment clients for next marketing campaign in one luxury goods company. From this point of view, *K*-means is used to find out the most significant clients of company through clustering. The main goal is to identify relevant clients, who are also loyal, and to use their profile to create new digital marketing campaigns [11-13].
The database of company has been observed with data between the December 2010 and April 2018 year. There are more than 9.000 customer purchases records in data base, more than 1200

customer interaction records, and more than 250 distinct customers. An algorithm is written in MATLAB, including the results and interpretation.

Clustering methodology proposed in this paper includes: loading and cleaning data (removing nulls, NaNs and removing outliers from database), preprocessing of data (reformatting of dates, indexing labels...), data analysis in order to find candidates for good quality features (visualize data), splitting data set on test and train sets, variable clustering to remove features with similar impact, testing and statistical comparison of candidates, feature vector normalization (mapping the distribution into 0.0–1.0 range), multiple clustering model testing with the selected features, and best model selection and final clustering using PCA (Principal Component Analysis) which is applied in order to reduce the dimensionality identifying patterns in data based on the correlation between features [14, 15]. Nearest neighborhood method [16] is used as linkage clustering method, while the Silhouette index [17] is used to profile clusters and to indicate the clusters' separation, using the Euclidean or some another distance metric.

The main target and result is to attract new clients based on analyzed profiles and behavior patterns. Thus, the desired profile of the company's possible clients will be created from the data on existing loyal clients. As a result, the company management team will be able to create a digital marketing campaign that will target exactly this market segment, or to create alternative campaigns for the other significant client segments as well.

Algorithm objectives and some optimization strategies

In this paper, the clients are grouped into five clusters:

Cluster1: Medium interaction shopaholic customers (smaller spenders who prefer high-index types of items and who are contacted a moderate amount of time),

Cluster2: Low interaction modest customers (smaller spenders who prefer high-index types of items and who are contacted a fairly low amount of times. They have a significant amount of purchases),

Clustre3: High interaction rich customers (big spenders who are contacted many times with high-index types of contact, but who prefer low-index types of items),

Clustre4: High interaction modest customers (smaller spenders who prefer high-index types of items and who are contacted many times),

Cluster5: Low interaction rich customers (big spenders who are contacted moderately with low-index types of contact and who prefer low-index types of items).

Let X_τ = $\{X_1,...X_N\}$ be the set of data points, where $C=(C_1,...,C_K)$ presents clustering into K groups. Let $d(X_k, X_l)$ be the Euclidean distance between X_k and X_l. Let $C_j = \left\{ X_1^j, ..., X_{m_j}^j \right\}$ present jth cluster, $j=1,\ldots,K$, where $m_j=|C_j|$. An average distance a_i^j between i-th vector in cluster C_j and other vectors in the same cluster is [4]:

$$a_i^j = \frac{1}{m_j - 1} \sum_{\substack{k=1 \\ k \neq i}}^{m_j} d\left(X_i^j, X_k^j \right), \quad i = 1, ..., m_j \qquad \textbf{(1)}$$

Minumum average distance between i-th vector in cluster C_j and all the vectors in cluster C_k, k=1,...,K, $k \neq j$, is:

$$b_i^j = \min_{\substack{n=1,\dots,K \\ n\neq j}} \left\{ \frac{1}{m_n} \sum_{\substack{k=1 \\ k\neq i}}^{m_n} d\left(X_i^j, X_k^n\right) \right\}, \quad i = 1,\dots,m_j \quad \textbf{(2)}$$

Silhouette index of i-th vector in cluster C_j is:

$$s_i^j = \frac{b_i^j - a_i^j}{\max\left(a_i^j,\, b_i^j\right)} \quad \textbf{(3)}$$

So, $-1 \le s_i^j \le 1$. Silhouette index of cluster C_j is:

$$S_j = \frac{1}{m_j} \sum_{j=1}^{m_j} s_i^j \quad \textbf{(4)}$$

while the global silhouette value is:

$$S = \frac{1}{K} \sum_{j=1}^{K} S_j \quad \textbf{(5)}$$

taking values from -1 to 1.

The Silhouette plot [17] shows that the data is split into K clusters, while the silhouette index would be calculated according to (5). Clusters are formed such that objects in the same cluster are similar, and objects in different clusters are distinct. K-means clustering is a partitioning method that treats observations in data set as objects having locations and distances from each other. It partitions the objects into K mutually exclusive clusters, such that objects within each cluster are as close to each other as possible, and as far from objects in other clusters as possible. Each cluster is characterized by its centroid, or its center point, while an iterative algorithm that assigns objects to clusters is used, so that the sum of distances from each object to its cluster centroid, over all clusters, is a minimum. At each iteration, the algorithm reassigns points among clusters to decrease the sum of point-to-centroid distances, and then recomputes cluster centroids for the new cluster assignments. In this paper, the Euclidean distance, the squared Euclidean distance and cosine distance are used.

Principal Component Analysis (PCA) is also applied, as a technique that is widely used for applications such as dimensionality reduction, visualization and lossy data compression. PCA can be defined as the orthogonal linear transformation of the data to a lower dimensional linear space, known as the principal subspace, such that the greatest variance by any projection of the data comes to lie on the first coordinate, called the first principal component, while the second greatest variance by any projection of the data comes to lie on the second coordinate, and so on [14]. PCA finds a meaningful coordinate basis to express the data. In this paper, PCA is used to transform data to a new basis and also to project them to a lower dimension.

Some experimental results

The Silhouette plot illustrated in Fig.1 shows that the data is split into five clusters, having large silhouette index values (0.7 or greater), indicating that the clusters are well separated. Silhouette index is calculated according to (5).

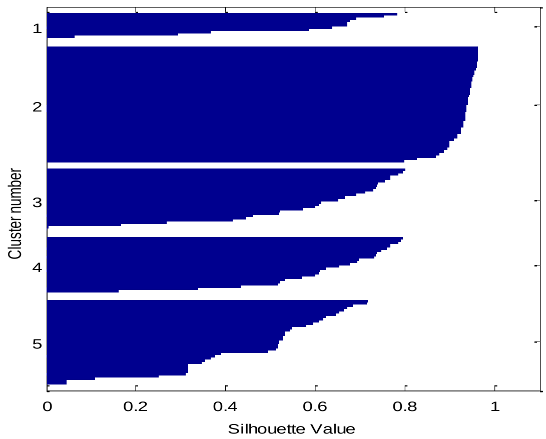

Fig. 2. Silhouette index value

Grouping into clusters, with visualization, following clients' distribution according to their attributes is given in Fig.2, Fig.3, Fig.4, Fig.5, and Fig.6, respectively.

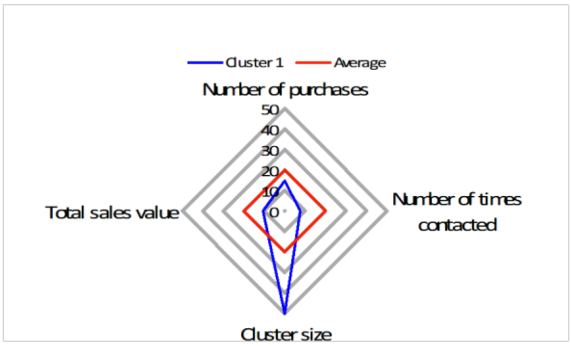

Fig. 2. Cluster1

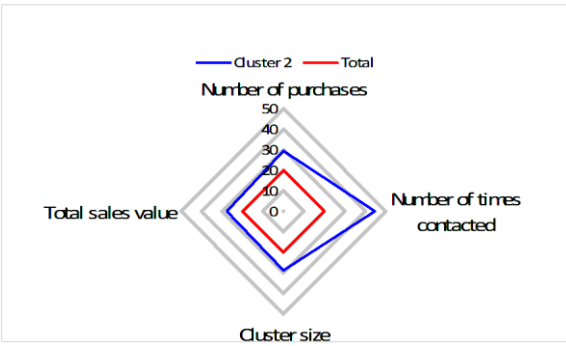

Fig. 3. Cluster2

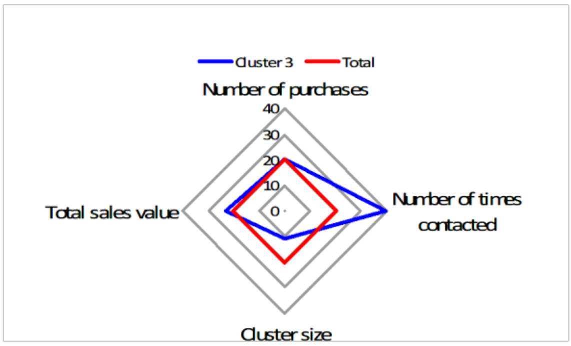

Fig. 4. Cluster3

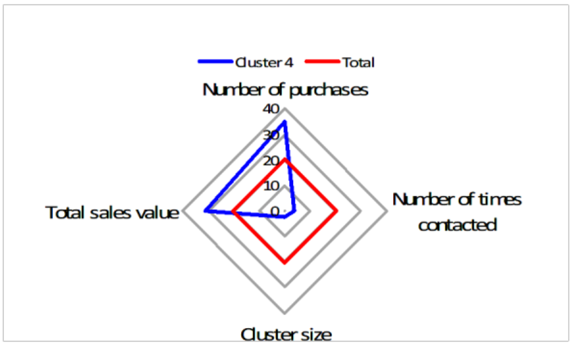

Fig. 5. Cluster4

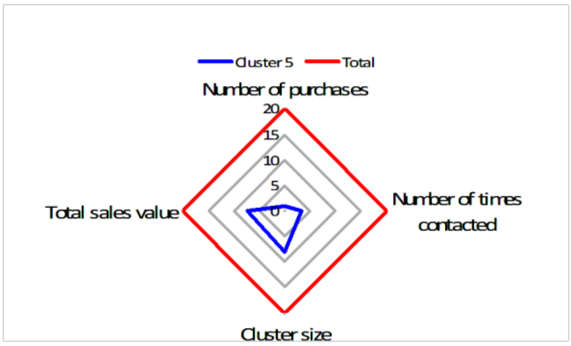

Fig. 6. Cluster5

Transforming data to a new basis using PCA in order to project them to a lower dimension is shown in Fig.7, illustrating an optimal partition of data samples in principal directions. The principal components of the new subspace can be interpreted as the directions of maximum variance given the constraint that the new feature axes are orthogonal to each other.

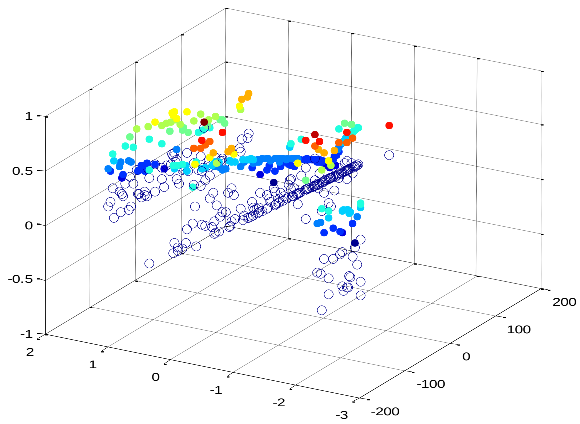

Fig. 7. Reduction of dimensionality performed by PCA

The relationship between attributes (number of purchases, number of times contacted, total sales value) with data points categorized into five clusters, presented in different colors, are shown in Fig.8 and Fig.9.

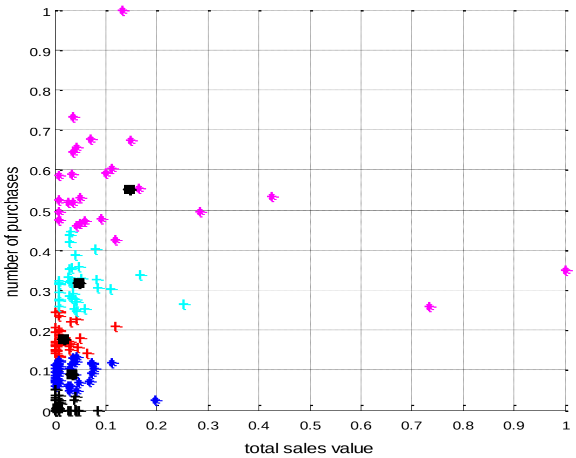

Fig. 8. Data points in number of purchase vs total sales graph

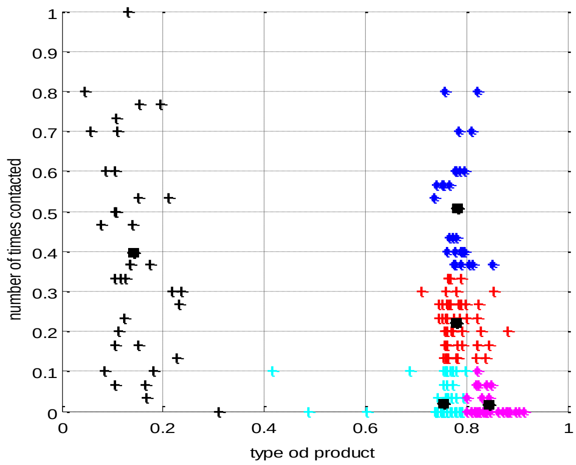

Fig. 9. Data points in number of purchase vs total sales graph

Conclusion

This paper contains description and demonstration of simple MATLAB-based *K*-means algorithm, used to iteratively re-assign clients to the nearest cluster center, with randomly selected *K* points as initial cluster center. *K*-means iterative procedure converges to local minima, but this local minimum is highly selective to the selected initial partition, so the selected initial partition is estimated applying principal component analysis. Results given in this paper could help in attracting and keeping clients, and it also allows company to better understand its clients, the market in which they are active, their competitors, and other factors that can impact their own business and profitability.

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