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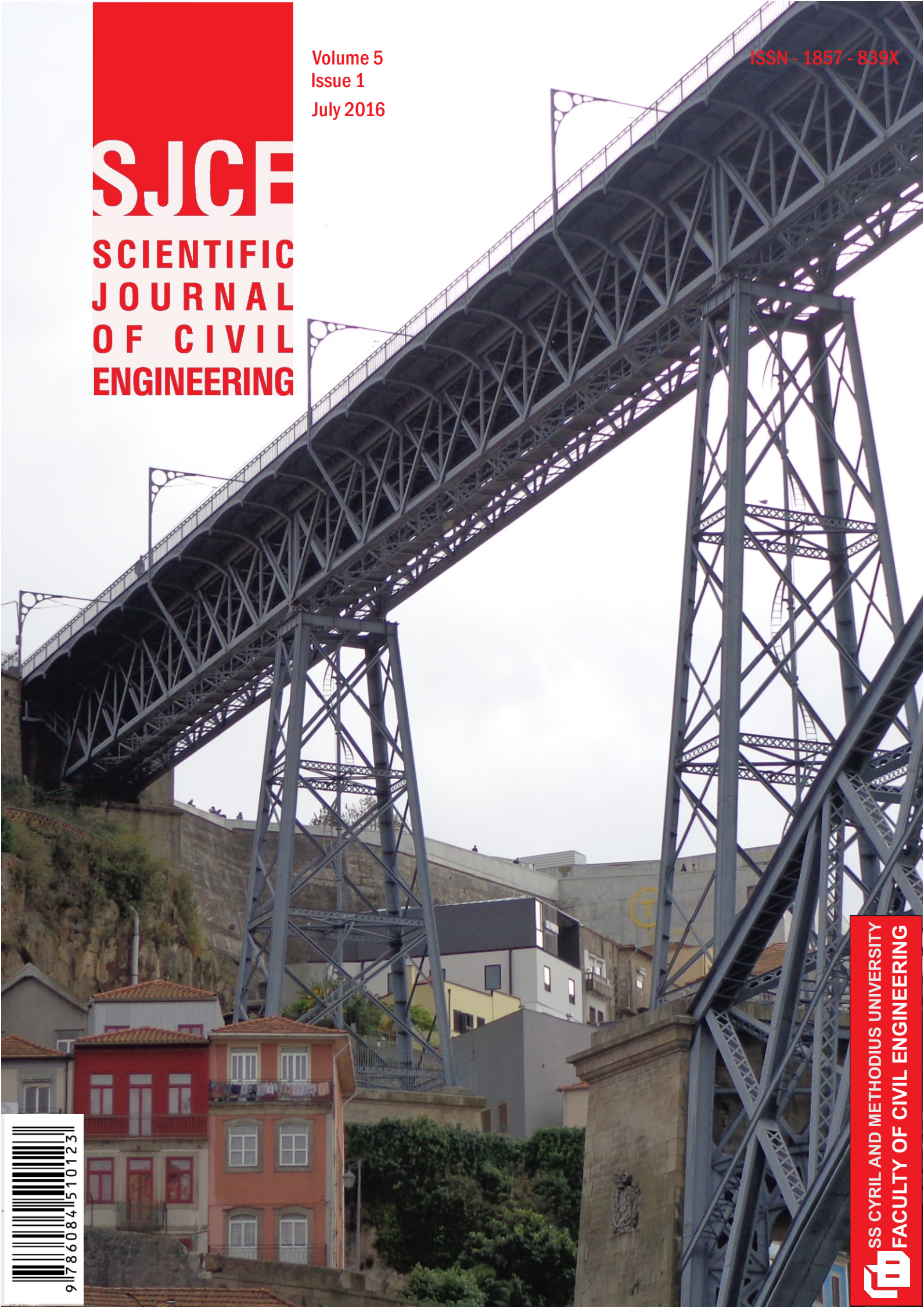

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EDITORIAL - Preface to Volume 5 Issue 1 of the Scientific Journal of Civil Engineering (SJCE)

Todorka Samardzioska EDITOR – IN - CHIEF

Dear Readers,

Scientific Journal of Civil Engineering (SJCE) was established in December 2012. It is published bi-annually and is available online at the web site of the Faculty of Civil Engineering in Skopje (www.gf.ukim.edu.mk).

This Journal welcomes original works within the field of civil engineering, which includes: all the types of engineering structures and materials, water engineering, geo-technics, highway and railroad engineering, survey and geo-spatial engineering, buildings and environmental protection, construction management and many others. The Journal focuses on analysis, experimental work, theory, practice and computational studies in the fields.

The international editorial board encourages all researchers, practitioners and members of the academic community to submit papers and contribute for the development and maintenance of the quality of the SJCE journal.

As an editor of the Scientific Journal of Civil Engineering (SJCE), it is my pleasure to introduce the First Issue of VOLUME 5. The primary goal continues to be high quality of publications, enhancing objectivity and fairness of the review process.

This issue includes 5 articles in the field of: steel, timber and composite structures, structural health monitoring, as well as implementation of ISO standardization in geodesy. The first paper provides detailed description of different collapse modes and presents methods for collapse mode determination of eccentrically patch loaded steel I-girder. The second paper represents a review of existing and innovative solutions of the cross laminated timber and its behaviour under seismic loading conditions, while the third one describes an implementation of structural health monitoring strategies for assessment of structural behaviour of steel chimney structure.

Comparison of experimentally tested behaviour of two different composite continuous beams has been explained in the fourth article. The final paper represents an implementation of systems for quality management in geodetic companies and the benefits of proper implementation and application of the standards are emphasised.

Wishing you nice summer and successful completion of the academic year!

Sincerely Yours,

Assoc. Prof. Dr. Todorka Samardzioska

June, 2016

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COLLAPSE MODES OF ECCENTRICALLY PATCH LOADED STEEL I-GIRDERS

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Patch loading acts locally, over a small area or length of a structural element. Some eccentricity of load relative to the web plane is unavoidable in engineering practice. Experiments show that most (but not all and not always!) eccentrically patch loaded steel I-girders have a collapse mode quite different from that of centrically loaded girders – *eccentric collapse mode*. However, in certain conditions, eccentrically loaded girders behave as if there is no load eccentricity, having *centric collapse mode*. *Mixed collapse mode*, with the characteristics of both centric an eccentric collapse modes, may also occur in eccentrically loaded girder.

Concerning engineering practice, the most important difference between collapse modes is in ultimate load. The reduction in ultimate load with an increase in load eccentricity is obvious in eccentric collapse mode, as well as in mixed collapse modes. Hence, in order to estimate ultimate load, the first step is to determine collapse mode of eccentrically patch loaded I-girder.

The paper provides detailed description of different collapse modes and presents methods for collapse mode determination – experimental based empirical criteria and ANN forecast models.

Keywords: steel I-girder, patch load, load eccentricity, collapse mode, eccentric, centric, mixed, collapse load.

1. INTRODUCTION

Patch load acts locally, over a small area or length of a structural element, Figure 1. Particularly intriguing problem and quite common situation in structural engineering is the case when local compressive load affects the flange of a steel I-profile so that the web, below the loading, is locally pressed. Local stresses in the vicinity of load might cause local instability that may provoke element carrying capacity loss and, consequently, collapse of the whole structure. This is rather complex and challenging issue of extremely evident elastic-plastic stresses and deformations. Apart from that, geometrical nonlinearity is noticeable even at the lowest loading level.

Patch loaded girders (Figure 1) are widely used in engineering practice: crane girders loaded by the crane wheels, bridge girders erected by launching etc.

Although some eccentricity of load relative to the web plane is unavoidable in engineering practice, rather modest amount of worldwide research work has treated this issue in comparison with the amount of worldwide research work that have treated centric patch loading. While over 35 experimental researches (with more than 750 tested samples) dealt with I-girders patch loaded in the web plane, influence of load eccentricity was analysed in only eight experimental studies (with less than 200 tested samples), summarised in [6]. Experimental analysis of eccentrically patch loaded girders started at the University of Maine in late 1980s. At the same time some tests were done at the Institute of Theoretical and Applied Mechanics, Czech Academy of Sciences. Ten years later a new series of experiments (1998, 2001, 2007) were initiated at the University of Montenegro [2,4,6]. The newest testing, with only four tested girders, was done at the University of Navarra, upon the initiative of the University of Granada, in 2009 [5]. Experimental work was followed by finite element method (FEM) modelling, by means of various computer software [1,4-6]. While over 30 mathematical expressions (mostly based on collapse mechanism) for centric ultimate load might be found in literature, only few empirical expressions for eccentric ultimate load have been proposed [1,5-7]. Artificial neural networks, being suitable for multi-parameter analysis, are also used nowadays for collapse mode and ultimate load estimation [6,7].

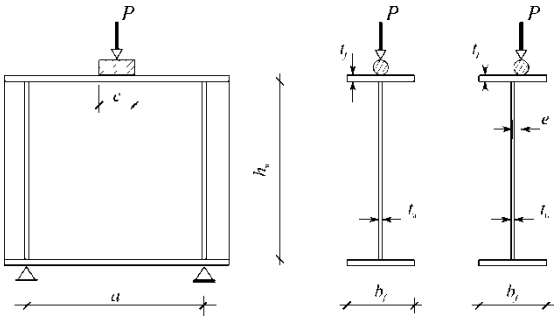
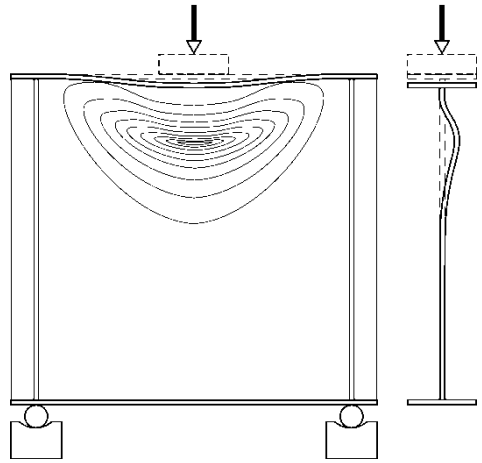


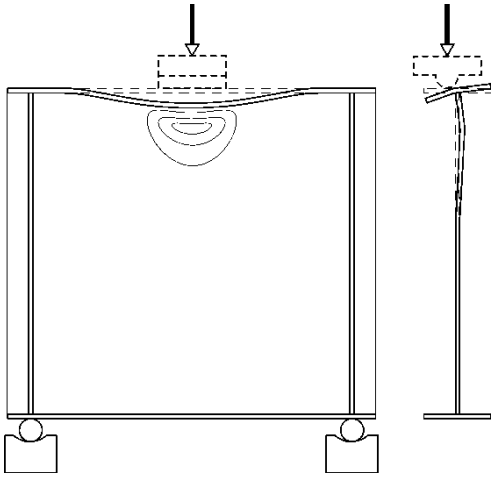
Figure 1. Centrically and eccentrically patch loaded steel I-girder

Since the first experimental investigations, in 1980s, it has been evident that numerous parameters influence the behaviour, collapse mode and ultimate load of eccentrically patch loaded steel I-girders: geometric parameters (girder's dimensions and their dimensionless ratios), load eccentricity and its relations with girder dimensions, as well as the manner of load application (line or laterally distributed load). Dominant parameter is the load eccentricity, e, or ratio e/b_f. Apart from this parameter, the influence of girder geometry parameters should be studied. Girder dimensions, primarily web and flange thicknesses, t_w and t_f, as well as ratio t_f/t_w are of important influence. Other ratios, like b_f/t_f, a/t_w, h_w/t_w should be considered as well. Attention should also be paid to the load length, c, or ratio c/a and to the load application manner. Hence, not only that influential parameters are numerous, but they are also mutually dependant and related and therefore should not be considered separately. Combinations of certain parameters should be carefully analysed. Dealing with such a big number of correlated influential parameters makes determination of collapse mode and calculation of collapse load difficult tasks.

Even 1980s tests shown and all later experiments confirmed that most eccentrically patch loaded steel I-girders (but not all and not always!) have a collapse mode quite different from that of centrically loaded girders. Carrying capacity loss in the case of centric load is due to web buckling and local stability loss (*centric collapse mode*, Figure 2a). In the case of eccentric load, girders lose carrying capacity mostly due to local elastic-plastic bending (*eccentric collapse mode*, Figure 2b). Experimental tests from 2007 confirmed previous assumption regarding possibility of mixed collapse mode occurrence. *Mixed collapse mode*, having characteristics of both, centric and eccentric collapse modes, may appear in two variants: as *centric-mixed* or as *eccentric-mixed collapse mode*, depending on dominant collapse mode characteristics.



a/ centric collapse mode



b/ eccentric collapse mode

Figure 2. Collapse modes typical for centric and eccentric patch loading

Concerning engineering practice, the most important difference between collapse modes is in ultimate load. The reduction in ultimate load with an increase in load eccentricity is obvious in girders with eccentric collapse mode. For a certain girder geometry, even the smallest load eccentricity ($e = 5\ mm$, i.e. $e/b_f = 1/30$) reduced ultimate load over 40%. In case of centric collapse mode in eccentrically loaded girders, ultimate load does not change significantly with an increase in load eccentricity. Even for highest load eccentricity ($e = 25\ mm$, i.e. $e/b_f = 1/6$), girders of certain geometry behaved as if there was no load eccentricity.

Hence, in order to estimate ultimate load, the first step is to determine collapse mode of eccentrically patch loaded I-girder. Some conclusions were made through the analysis of experimental data and summarised in the form of criteria for collapse mode determination based on relations of geometrical parameters. Another approach that was analysed is application of artificial neural networks (ANN) as a highly suitable tool for dealing with numerous and mutually related influential parameters.

2. EXPERIMENTAL RESEARCHES AT THE UNIVERSITY OF MONTENEGRO

A series of experimental researches analysing patch loading problems was organised at the University of Montenegro, since 1998 [2,4,6]. Among the other issues, particular attention was paid to the problem of eccentric patch loading. Figure 3 shows tested girder with characteristic dimensions. Girder span, a, and web depth, h_w, as well as flange width, b_f, are same in all tested girders ($a = h_w = 700\ mm$, $b_f = 150\ mm$). Web thickness, t_w, flange thickness, t_f, as well as load eccentricity, e, are variables ($t_w = 3 \div 10\ mm$, $t_f = 3 \div 15\ mm$, $e = 0 \div 30\ mm$), making also variable influential parameters as e/b_f, b_f/t_f, t_f/t_w, h_w/t_w.

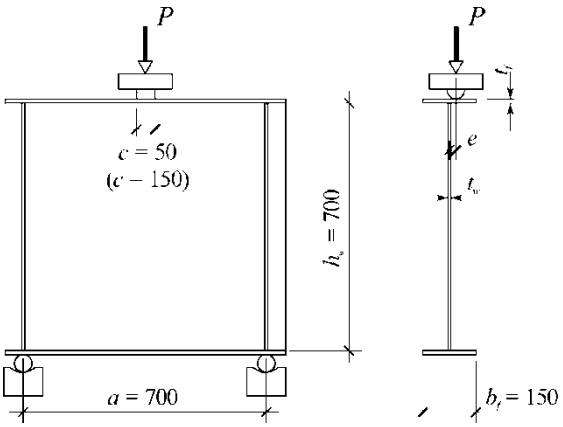


Figure 3. Experimentally tested I-girder under eccentric patch loading

For the analysis of eccentric patch loading, 144 tests were done in experiments from 1998 [2], 2001 [1,3-5,7] and 2007 [1,5-7]. Girders are grouped in series, by the dimensions. Six girders of same geometry make one series where load eccentricity varies in range from $e = 0$ to $e = b_f/6 = 25\ mm$ with an increment of $5\ mm$. A few girders were also tested with load eccentricity $e = 30\ mm$. Most girders were loaded over the length $c = 50\ mm$, while several samples had load length $c = 150\ mm$.

2.1. COLLAPSE MODES

Three different collapse modes are observed in experimentally tested eccentrically patch loaded steel I-girders: eccentric, centric and mixed collapse mode.

2.1.1. ECCENTRIC COLLAPSE MODE

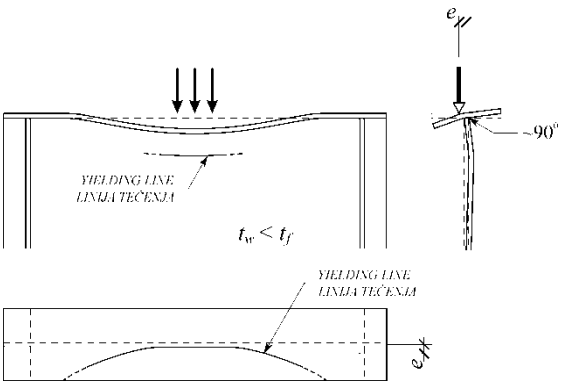
During the process of load increase, the most evident deformation in case of *eccentric collapse mode* (Figures 4 and 5) is the flange warping accompanied by gentle web bending which follows the flange deformation, due to flange-web constraining. The web thickness and the flange stiffness influence the extent of flange warping. The larger the eccentricity, the more pronounced the deformation is, especially in flange. Flange warping and web bending grow until the final collapse of girder.

A yielding line appears in the flange and surrounds the load, forming an arch, Figure 5. The length of arch chord primarily depends on the extent of eccentricity, and secondarily on the ratio t_f/t_w. As the eccentricity increases, the arch chord decreases almost linearly. The smaller the ratio t_f/t_w, the shorter the chord, i.e. the "shaper" the arch, Figures 5a, 5b. This flange yielding line is not quite clearly visible, i.e. it is not completely developed in girders with rather thick flange, Figure 5c. Apart from yielding line in the flange, there is a horizontal yielding line in the web. In cases of girders with thicker web, i.e. in cases with $t_w = t_f$, the web yielding line is far less evident, almost invisible, Figure 5b. It is also closer to the loaded flange than in case with $t_w < t_f$. Loading eccentricity influences the depth of yielding line in web, i.e. its distance from the loaded flange.

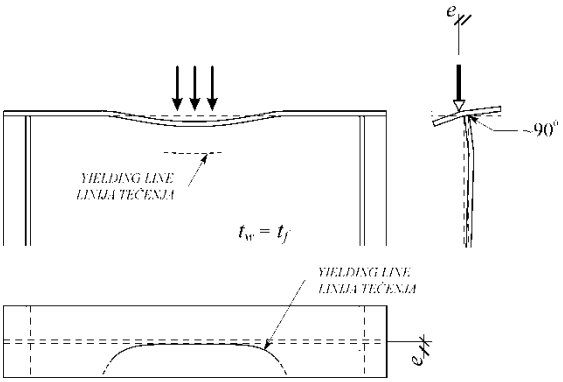
Figure 4. Eccentric collapse mode - test sample with t_w = 3 *mm*, t_f = 3 *mm*, e = 5 *mm*

The greater the eccentricity, the closer the yielding line gets to the loaded flange. The change of t_f/t_w ratio does not affect the depth of web yielding line appearance in girders with thinner web, i.e. in cases with $t_w < t_f$. It seems that initial imperfection does not influence girder behaviour.

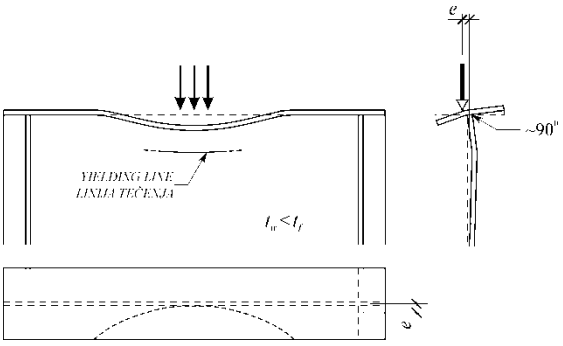
The most important characteristic of eccentric collapse mode is significant reduction of ultimate load due to load eccentricity. The reduction is greater for smaller t_f/t_w ratios.



a/ In girders with $t_w < t_f$ and $b_f/t_f \geq 12.5$ both yielding lines, in flange and in web, are clearly visible.



b/ In girders with $t_w = t_f$ and $b_f/t_f \geq 12.5$ yielding line in web is almost invisible.



c/ In girders with thick flange (b_f/t_f = 10) yielding line in flange is not clearly visible, i.e. it is not totally developed.

Figure 5. Eccentric collapse mode(s), typical of eccentrically patch loaded I-girders

2.1.2. CENTRIC COLLAPSE MODE

Although typical for centrically loaded girders, *centric collapse mode* (Figures 6-9) often happens in case of eccentric patch load. The smaller the eccentricity ratio, e/b_f, and the larger the t_f/t_w ratio (i.e. thinner web relative to flange), the more likely a centric collapse mode would develop in girders having eccentric patch loading. During the process of load increase, both flange and web deformations are insignificant until the collapse, which happens suddenly.

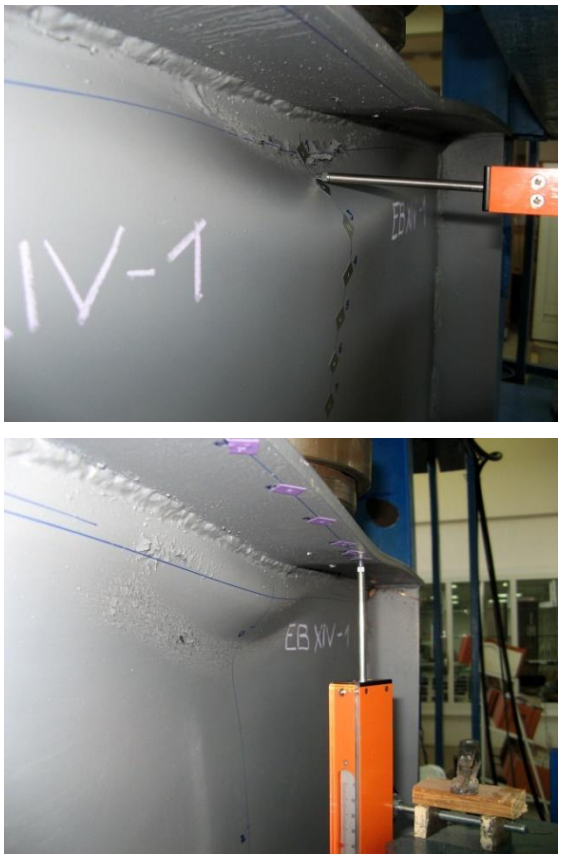


Figure 6. Centric collapse mode with two yielding lines in web - test sample with t_w = 4 *mm*, t_f = 8 *mm*, e = 0 *mm*

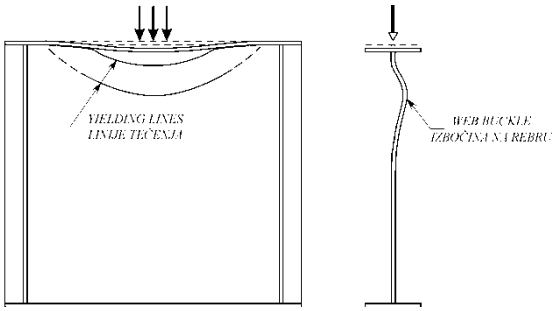


Figure 7. Centric collapse mode with two yielding lines in web

The loaded flange remains almost completely horizontal until the moment of collapse, when it moves downwards (below the loading), due to web crippling and/or buckling. Flange simply sinks, as it follows collapse deformation of web. A considerable web squashing is obvious in girders with thicker webs (t_w = 8 and 10 mm). The thicker the web, the more evident the squashing (yielding) becomes, before reaching failure load. However, apart from the obvious web squashing under the loading, the girder collapse is characterised by the occurrence of a crippling in the web.

Figure 8. Centric collapse mode with three yielding lines in web - t_w = 5 *mm*, t_f = 12 *mm*, e = 5 *mm*

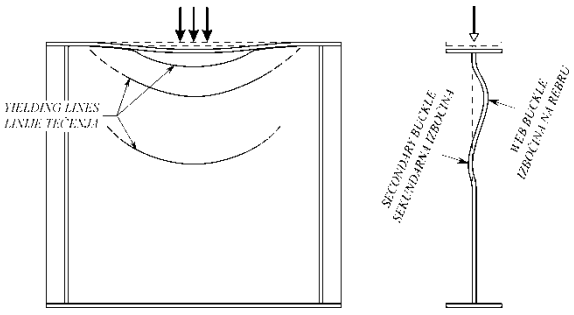


Figure 9. Centric collapse mode with three yielding lines in web

The noticeable web buckle is usually contoured by two yielding lines, Figures 6 and 7. The upper line slightly arches the load distribution block. This line divides the upper part of the web, which remains flat and perpendicular to the flange, from the lower part where the buckling is evident. The lower yielding line outlines the buckling form. Both web thickness and flange stiffness influence the web buckling form. The stiffer the flange, the closer (to the loaded flange) the upper yielding line gets, whereas the lower yielding line goes deeper into the web. The thicker the web is, the smaller the crippling area becomes (including both the width and the depth).. Several eccentrically loaded girders have slightly different centric collapse mode with three plastification lines in web, Figures 8 and 9. Immediately after primary collapse mechanism, formed by two yielding lines contouring buckle in the upper part of the web, in secondary mechanism the third yielding line appears deeper in the web, below previously formed two lines, in the middle or lower part of the web. The third line contours one more, smooth, secondary buckle in the web.

There is almost no change in ultimate load with an increase in load eccentricity in case of centric collapse mode.

2.1.3. MIXED COLLAPSE MODE

Mixed collapse mode occurs in two varieties – as *centric-mixed collapse mode* (Figures 10 and 11) or *eccentric-mixed collapse mode* (Figures 12 and 13). Mixed collapse modes have characteristics of both, centric and eccentric collapse modes, with one of them as dominant. The dominance determines type and full name of mixed collapse mode. In both types of mixed collapse mode, ultimate load reduces somewhat with the increase in load eccentricity. The ultimate load reduction is less manifest in mixed collapse mode than in eccentric collapse mode.

In the case of *centric-mixed collapse mode*, collapse happens due to web buckling in basic centric collapse mechanism, with two yielding lines contouring the buckle in web, Figures 10 and 11. However, gentle but growing torsion of flange is noticeable during the load application. It is caused by load eccentricity or by initial geometrical imperfection. The flange torsion is not emphasised too much, but it is obvious in residual deformation, even though both sides of flange, the loaded side, as well as the one on the opposite side of the web, are moved downwards. Although torsionally deformed almost all the time, in the moment of collapse flange follows the web buckling deformation and it sinks into the buckled web.

Figure 10. Centric-mixed collapse mode - test sample with t_w = 4 *mm*, t_f = 10 *mm*, *e* = 15 *mm*

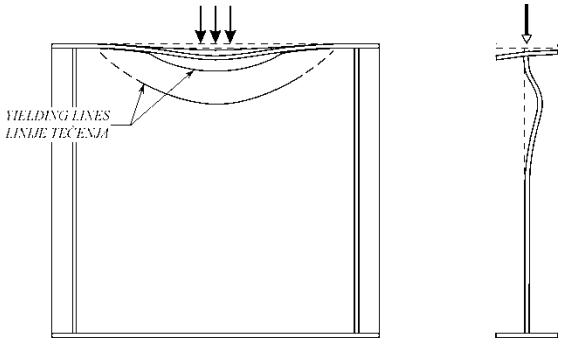


Figure 11. Centric-mixed collapse mode with gentle torsion of flange and centric collapse of web

In the case of *eccentric-mixed collapse mode*, collapse mechanism is not precisely defined, Figures 12 and 13. It might not be said surely if it is centric collapse of web, due to buckling, or eccentric collapse mode, due to bending. One (upper) plastification line in web is visible. The other one (lower) is not clearly visible. The question is if the lower yielding line occurs in the primary collapse mechanism, or in the secondary mechanism, immediately after the primary collapse of girder. In addition, it might not be said if plastification really happens along the lower line and at which length.

Figure 12. Eccentric-mixed collapse mode - test sample with t_w = 4 *mm*, t_f = 10 *mm*, e = 20 *mm*

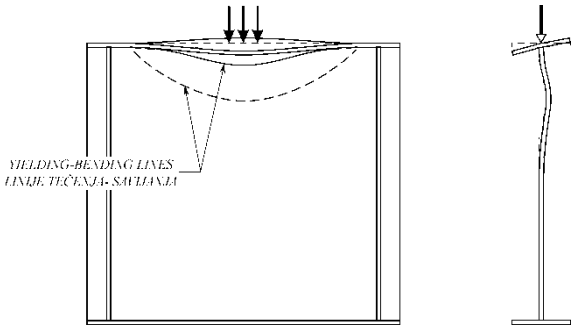


Figure 13. Eccentric-mixed collapse mode with pronounced torsion of flange and undefined web collapse

Buckle in the web is noticeable in some girders, but it is smooth and much less evident than in case of centric collapse mode. The flange is in torsion, due to load eccentricity and/or initial imperfection. The torsion propagates during the loading process and remains obvious in the residual deformation, with loaded side of flange moved downwards and the back side (part of the flange on the opposite side of the web, regarding the load eccentricity) moved upwards.

2.2. COLLAPSE MODE DETERMINATION

Occurrence of a certain collapse mode depends on load eccentricity and girder geometry. Web and flange thicknesses, t_w and t_f, are girder dimensions of the most significant influence. The most influential dimensionless geometry parameter is ratio t_f/t_w. Influence of web slenderness, i.e. parameter $a/t_w = h_w/t_w$ is also important. Actually, parameters t_f/t_w and $a/t_w = h_w/t_w$ should not be analysed separately. Both parameters together, i.e. their combination should be considered. Furthermore, correlation with the eccentricity ratio e/b_f should be established. Apart from ratio e/b_f, two more variants of dimensionless eccentricity might be considered: e/t_f and e/t_w.

Based on the experimental data from Montenegrin tests [i.e. girder dimensions in range:

- 5 *mm* ≤ e ≤ 30 *mm*,
- 3 *mm* ≤ t_w ≤ 10 *mm*,
- 3 *mm* ≤ t_f ≤ 15 *mm*,
- $a = h_w$ = 700 *mm*, b_f = 150 *mm*, c = 50 or 150 *mm*;

and dimensionless parameters in range:

- (0) 1/30 ≤ e/b_f ≤ 1/5,
- (0) 0.33 ≤ e/t_f ≤ 8.33,
- (0) 0.50 ≤ e/t_w ≤ 8.33,
- 1 ≤ t_f/t_w ≤ 5,

- $10 \le b_f/t_f \le 50$,
- $70 \le a/t_w = h_w/t_w \le 233$,
- $a/h_w = 1$, $c/a = 0.071$ or 0.214],

collapse mode determination criteria summarised in Table 1 have been established. Girders fulfilling criteria in dark grey fields have eccentric collapse mode, with the reduced ultimate load. Girders fulfilling criteria in light grey fields might have any collapse mode and these criteria imply that such girders should be carefully analysed each separately, with its specific characteristics, considering several influential parameters and their combinations, as well as paying particular attention to the initial deformation.

Collapse mode determination criteria from Table 1 correlate dimensionless parameters t_f/t_w and $a/t_w = h_w/t_w$ with dimensionless eccentricity e/b_f, e/t_f or e/t_w. It is found out that primary influential parameter t_f/t_w enables quality identification of collapse mode in correlation with each one of the three dimensionless eccentricities. On the other side, web slenderness $a/t_w = h_w/t_w$ correlated with any of the three dimensionless eccentricities does not enable complete and fully reliable collapse mode determination. Correlations of dimensionless parameters from Table 1 provide high level of reliability in collapse mode determination. Combining (checking) several of these criteria increases the reliability level.

Some girder dimensions or dimensionless parameters, analysed separately, without making relation(s) with the other parameter(s), may be used as a rough identifiers of collapse mode. Girders having thin flange ($t_f \le 6$ mm) or thick web ($t_w \ge 8$ mm) or low web slenderness ($h_w/t_w < 100$) or ratio $t_f/t_w < 2$ are affected even with the lowest eccentricity. On the other side, girders with the ratio $t_f/t_w > 3$ have centric collapse mode even at the highest eccentricity. Obviously, parameter t_f/t_w has the key-role nevertheless if analysed in correlation with the other parameters or separately, only by itself.

3. ANN FORECAST MODELS FOR COLLAPSE MODE

Although the problems of collapse mode determination and collapse load calculation are mutually connected, collapse mode quantifying the level of collapse load, at the initial phase of ANN modelling these two issues are considered independently from each other. At first, artificial neural network modelling was implemented in estimation of collapse load, without considering collapse mode. Later on, the same networks, that gave good results in estimation of collapse load, were slightly modified, adapted and used to forecast collapse mode. The only one computer software [6] was used for training of all created artificial neural networks.

Table 1. Dimensionless parameter correlation criteria for collapse mode determination

dimensionless eccentricity	correlated dimensionless parameter	criterion	collapse mode
e/b_f	h_w/t_w	$h_w/t_w < 1050 \cdot e/b_f + 35$	E
		$h_w/t_w \ge 1050 \cdot e/b_f + 35$	E, C, M
	t_f/t_w	$t_f/t_w < 15 \cdot e/b_f + 0.5$	E
		$15 \cdot e/b_f + 0.5 \le t_f/t_w \le 15 \cdot e/b_f + 1.5$	E, C, M
		$t_f/t_w > 15 \cdot e/b_f + 1.5$	C
e/t_f	h_w/t_w	$h_w/t_w \le 85 \cdot e/t_f + 60$	E
		$h_w/t_w \ge 85 \cdot e/t_f + 70$	C
		$85 \cdot e/t_f + 25 \le h_w/t_w \le 85 \cdot e/t_f + 105$	M
	t_f/t_w	$t_f/t_w < e/t_f + 0.5$	E
		$e/t_f + 0.5 \le t_f/t_w \le e/t_f + 1.7$	E, C, M
		$t_f/t_w > e/t_f + 1.7$	C
e/t_w	t_f/t_w	$t_f/t_w < 0.3 \cdot e/t_w + 0.8$	E
		$0.3 \cdot e/t_w + 0.8 \le t_f/t_w \le 0.3 \cdot e/t_w + 1.8$	E, C, M
		$t_f/t_w > 0.3 \cdot e/t_w + 1.8$	C

The basic idea is to estimate the collapse load and/or collapse mode as the only output parameter, depending on numerous input parameters (material characteristics, girder geometry and load eccentricity), as well as to asses applicability of ANN modelling method for collapse load/mode determination in engineering practice.

ANN models were made separately for girders with different load lengths (c = 50 or 150 *mm*). Herein only load length of $c = 50$ *mm* and models with five dimensional inputs (e, t_w, t_f, $\sigma_{0.2,w}$ – web yielding stress, $\sigma_{0.2,f}$ – flange yielding stress) and one output (collapse mode) are considered. In order to use available software [6] for network training, it was necessary to transform collapse mode, being alpha-numerical, descriptive parameter, in the form of numerical parameter. Each type of collapse mode was assigned a numerical value from the interval [-1, 1]: centric collapse mode = 1; centric-mixed collapse mode = 0.3; eccentric-mixed collapse mode = -0.3; eccentric collapse mode = -1.

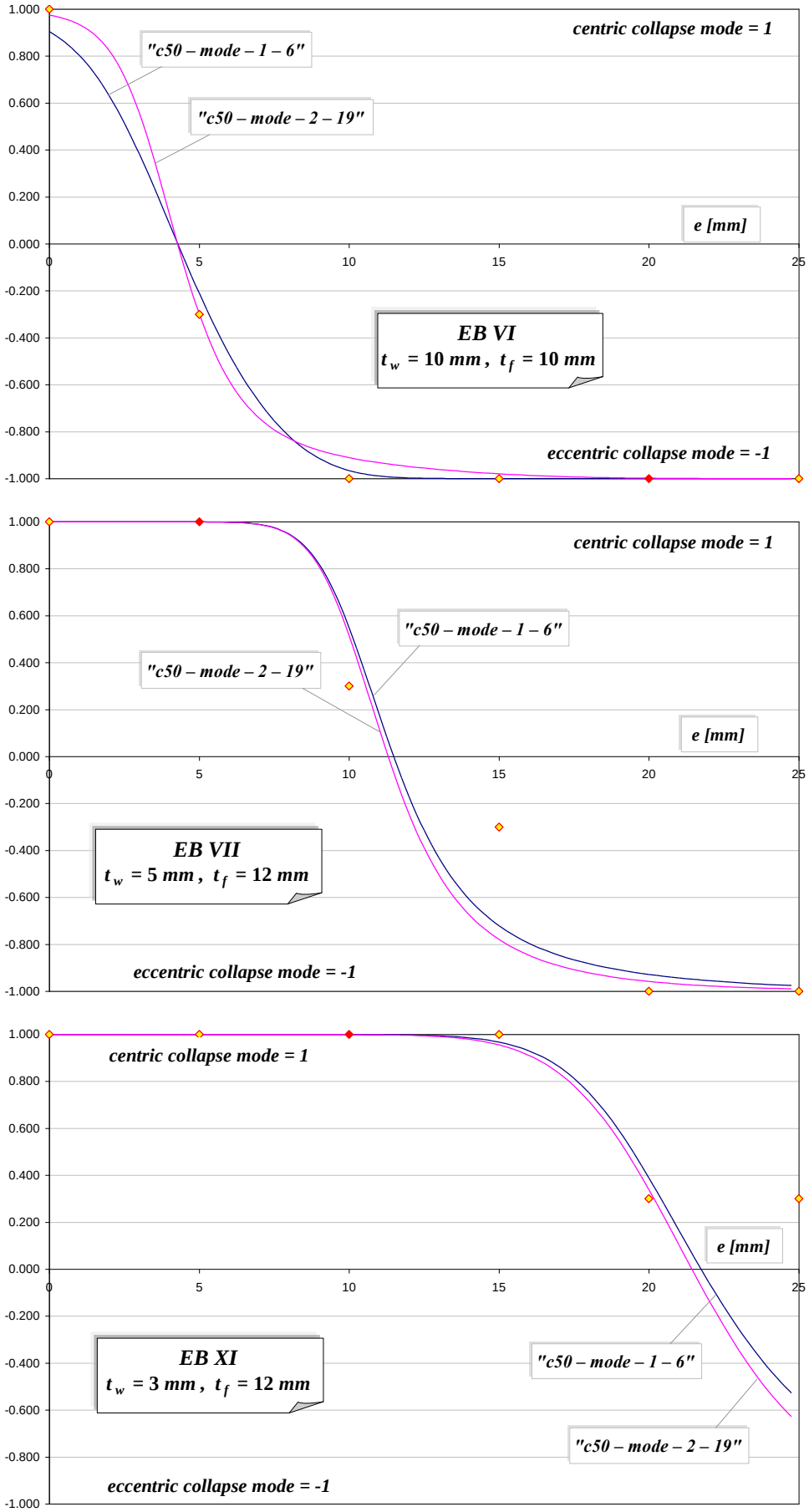
The complete experimental data set for girders with load length $c = 50$ *mm* consists of 120 tested girders. 19 testes were exempted from the network training process and used as a comparison data set, i.e. as data for the evaluation of forecast models. The rest of 101 tests were divided in training data set (71 tests) and validation data set (30 tests).

Among created networks, the best results are obtained from network with one hidden level, with six neurons ("*c50-mode-1-6*"), and from network with two hidden levels, each with 19 neurons ("*c50-mode-2-19*"). Examples of collapse mode forecast models of these two networks are presented in Figure 14. Four characteristic girder geometry types, i.e. four experimental series having representative sample in comparison data set were chosen for this presentation and comparison of ANN modelling of collapse mode with experimental results. In Figure 14, lines present ANN forecast models for collapse mode, while dots present experimental results. Red dot is sample from comparison data set. Yellow dots are samples of same geometry, but with different load eccentricity. Each of four diagrams presents one complete experimental series.

Generally speaking, presented ANN forecast models match experimental data well, what makes them applicable in practice. Noticeable discrepancy is observed only in case of centric collapse at high eccentricity of load, Figures 14c and 14d. It happens in girders with high ratio t_f/t_w. Only collapse mode for highest eccentricity (e = 25 *mm*) is not well forecasted, while ANN models provide good forecast of collapse mode for other eccentricities ($e \leq 20$ *mm*) even in these girders.

Figure 14 illustrates very well the statement concluded from the experiment: “The smaller the eccentricity ratio, e/b_f, and the larger the t_f/t_w ratio (i.e. thinner web relative to flange), the more likely a centric collapse mode would develop in girders having eccentric patch loading.”, Chapter 2.1.2. of the paper. Trend of increase in possibility of centric collapse mode with an increase of t_f/t_w ratio from 10/10 = 1 (Figure 14a), over 12/5 = 2.4 (Figure 14b) and 12/3 = 4 (Figure 14c) to 15/3 = 5 (Figure 14d) is obvious. No centric collapse mode even for smallest eccentricities in case of low values t_f/t_w. However, even largest eccentricities will result in centric collapse mode in case of high values of t_f/t_w.

Similar diagrams, presenting other influential parameters, may illustrate other interesting dependences and lead to important conclusions concerning behaviour of eccentrically patch loaded steel I-girders.


centric collapse mode = 1
"c50 – mode – 1 – 6"
"c50 – mode – 2 – 19"
e [mm]
EB VI
t_w = 10 mm , t_f = 10 mm
eccentric collapse mode = -1
centric collapse mode = 1
"c50 – mode – 1 – 6"
"c50 – mode – 2 – 19"
e [mm]
EB VII
t_w = 5 mm , t_f = 12 mm
eccentric collapse mode = -1
centric collapse mode = 1
e [mm]
"c50 – mode – 1 – 6"
"c50 – mode – 2 – 19"
EB XI
t_w = 3 mm , t_f = 12 mm
eccentric collapse mode = -1

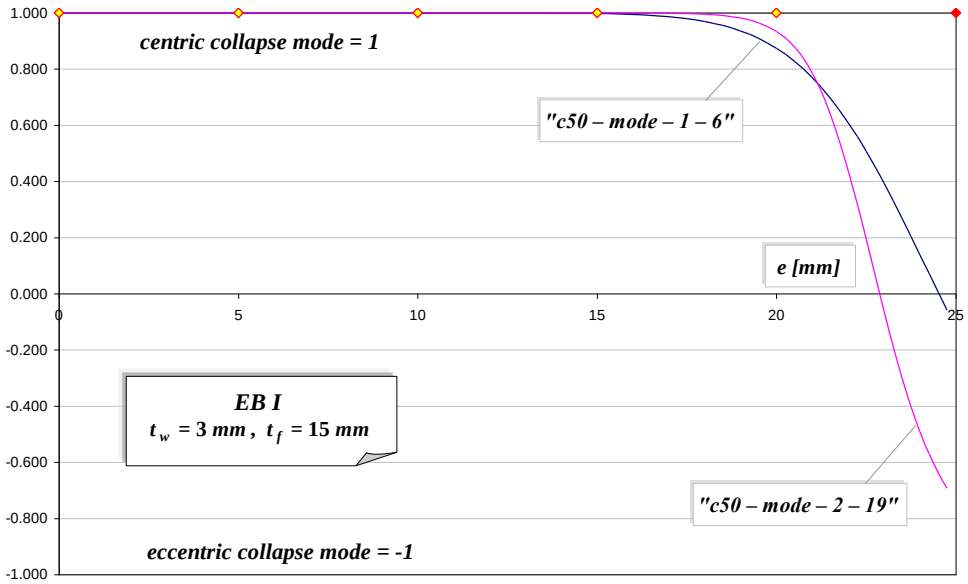


Figure 14. Examples of collapse mode forecast models

4. CONCLUSIONS

Existing experimental database, significantly enriched by Montenegrin tests from 1998, 2001 and 2007, enabled formulation of empirical criteria for collapse mode determination. Proposed criteria are valid for the domain of experimental data. The question is: what about girders with the dimensions out of experimental data domain? Graphical presentation of ANN forecast models (Figure 14) might help answering this question. Diagrams in Figure 14 imply separation of collapse mode types in girder of certain geometry, depending on load eccentricity. Zones of centric, mixed and eccentric collapse modes might be identified. By means of such diagrams, adequate ANN forecast models (although also connected with and resulted from the experimental database) might help in establishing new, general criteria for collapse mode identification, as well as their validation out of the experimental domain.

Both presented methods already in the existing form prove possibility of successful and useful application in engineering practice for the purpose of determination of collapse mode type in eccentrically patch loaded steel I-girders. Their parallel application may provide even more reliable results. However, further development of ANN forecast models should be welcomed, not only because of improvement of this method for itself, but also for its comprehensive benefit to the subject issue.

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STATE-OF-THE-ART REVIEW ON SEISIMIC BEHAVIOUR OF XLAM STRUCTURES

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Cross laminated timber is a novel engineered wood product with light weight, high strength, good thermal properties, relatively good fire resistance as well as fast onsite construction. Since its appearance in the European market, about 15 years ago, it becomes very desirable material for design and construction.

This paper reviews previous work relevant to cross laminated timber's behaviour under seismic loading conditions. Additionally, new innovative solutions for XLam applications have been presented.

Keywords: Cross laminated timber, seismic response, innovative applications

INTRODUCTION

In the last fifteen years, wood as a building material is coming back into play, mainly due to generally increased concern for the environment, the impact that materials are having on human health as well as rediscovered benefits of the natural materials. Practice also showed that wooden structures are extremely resistant to earthquakes, due to the small seismic force - wood is solid but light-weighted, so the energy generated in the wooden structures as a result of the ground acceleration is not as great as in structures made of other types of materials. Additionally, timber is more flexible than other types of materials, absorbing and dissipating energy. On the other hand, constant development of technology offers new production methods resulting in new sophisticated wood products. One of them is cross laminated timber.

Panel elements made of cross laminated timber or XLam, consist of three, five, seven or more layers of soft wood, mainly spruce, but pine or larch are also used, glued together along the entire surface of each panel, Fig. 1.

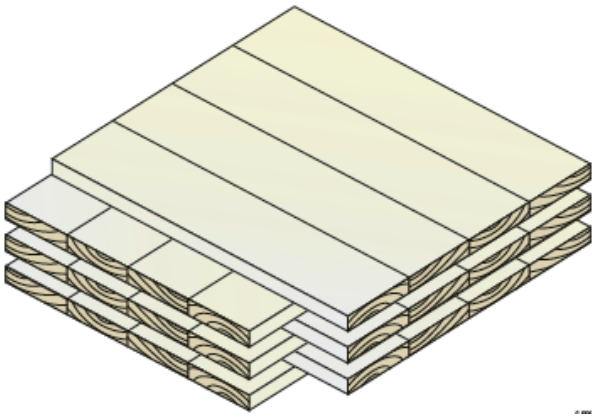


Figure 1. Configuration of XLam panel

The production process itself gives the panel element array of advantages compared to the standard timber elements used for construction. It minimizes swelling and shrinkage in the board plane, considerably increases static strength and shape retention properties and enables load transfer across the entire plane of panel. Also outer layers of the panel elements, that actually define the strength and appearance of the element, are made of wood with better quality that is wood with higher strength and more sophisticated surface in comparison to the inner layers, enabling in that way usage of wood with poorer quality. Size and shape of the panel elements are defined by each manufacturer, and depend of the transport and installation possibilities. The site installation is fast, simple and does not require especially skilled workers.

Generally, it can be concluded that Xlam structures become serious competition to traditional concrete or steel structures. However, there are still a lot of uncertainties regarding the characteristics and behaviour of this type of structures. As a result, these structures are not yet included in the current European standards for building and no guidelines or recommendations for their design and construction can be found.

SEISMIC RESISTANCE OF XLAM STRUCTURES

In conjunction with the rapid popularity of XLam structures in the construction market, there has been a parallel growth of researches and studies conducted in order to define their behaviour as well as their advantages and disadvantages. So far several investigations have been performed in Europe and North America aiming towards better understanding of the potential that XLam structures have, especially when it comes to their earthquake resistance.

SOFIE Project

Most comprehensive research was done by Prof. Ario Ceccotti from Italy and that is so called SOFIE project, realized by the Trees and Timber Institute of the National Research Council of Italy (CNR-IVALSA) in cooperation with the National Institute for Earth Science and Disaster Prevention (NIED), Shizuoka University and Building Research Institute (BRI) in Japan, partly supported by the Autonomous province of Trento, Italy.

The preformed experimental tests within the SOFIE project, include tests on single anchors, plane cyclic tests on Xlam shear wall elements with different disposition of connections and openings [1], pseudo-dynamic tests on 3D single storey model in three different configuration with different disposition of the openings [2], shaking table tests on a 3-storey building under few different earthquake records, [3] as well as shaking table tests on a seven storey building, Figure 2, [4].

The results from the tests showed that:

The response of the structural system depends mostly on the type and disposition of used connections. Connections represent the main source of nonlinearity and dissipated energy. Obtained hysteretic loops show equivalent viscous damping of 12% leading to conclusion that the observed structural system is rigid, but ductile. Figure 3 presents some of the observed failures pointing out that all forces and displacements are located in the relatively small part of the panel element resulting in only local damages, while the wooden part of the panel remains completely rigid.

Pseudo-dynamic tests also confirm those conclusions. Three different configurations regarding the openings in the panel elements have been tested, Figure 4:

- First configuration, external opening 1.20 m, internal opening 2.25 m;
- Second configuration, all openings are 2.25 m;

Third configuration, asymmetric opening 4 m, other openings 2.25 m.

In-plane cyclic tests

Pseudo-dynamic tests

Shaking-table tests on 3-storey model

Shaking-table tests on 7-storey model

Figure 2. Part of the tests performed within the SOFIE project

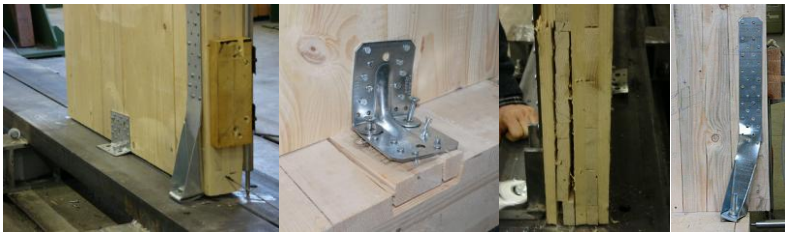


Figure 3. Local failures in different type of connections

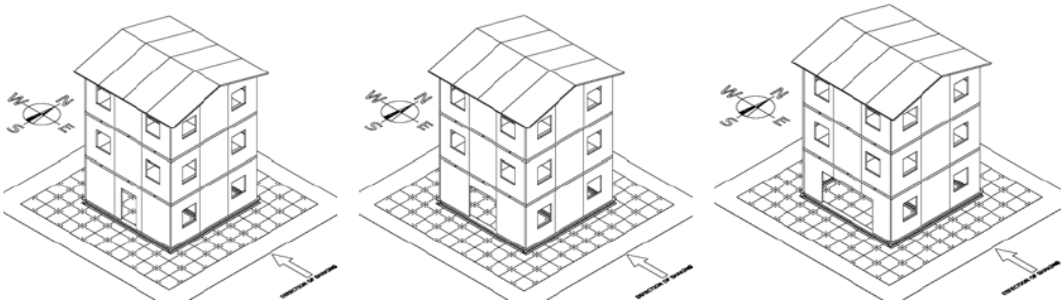
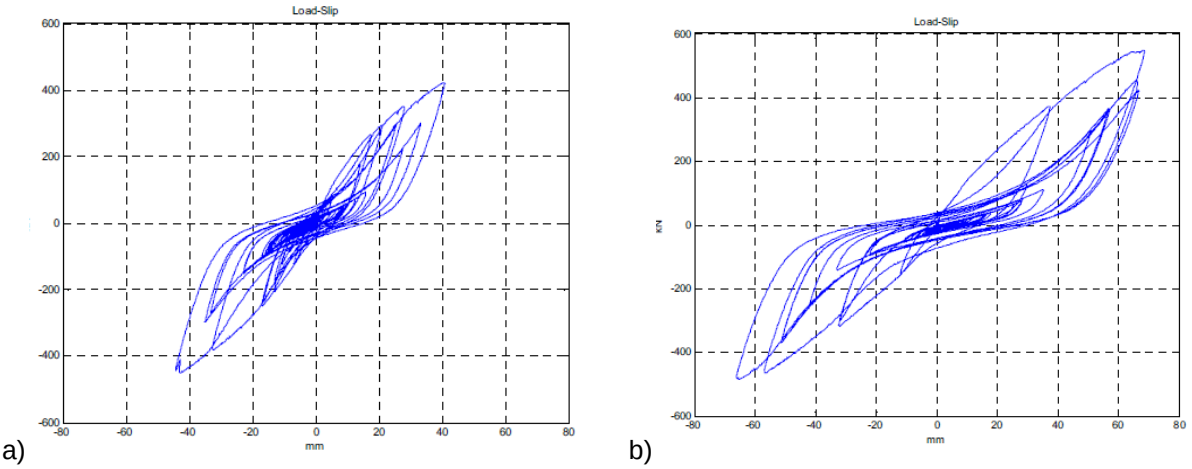


Figure 4. Different configurations of the tested 3-storey model

Obtained results, Figure 5, showed that the third configuration specimen with the asymmetric opening of 4m and the second configuration specimen with symmetric opening of 2.25m, have similar initial stiffness, meaning that the bigger opening does not influence the stiffness of the structure that is when it comes to lower values of the shear force, behaviour of the structure is mostly governed by the connections and not by the wooden panel. After 15 destructive earthquakes, the tested 3 storey building model did not show any fatal and no reparable damages. The near-collapse criteria, defined as failure of one or more holdown anchors, has been reached during the last test on the third configuration specimen preformed with the Nocera Umbra earthquake record with PGA (Peak Ground Acceleration) of 1.2g, Figure 6.



a) b)

Figure 5. Force-displacement diagram, Kobe JMA 0.5g, second and third configuration

Figure 6**.** Failure of the holdown anchor after Nocera Umbra, PGA=1.2g

By implementing specific subroutines, that account for the nonlinear behavior of the connections within the timber structures, in the DRAIN-3DX program, analytical model of the tested three storey building has been developed [5]. Comparison of the experimental and analytical results have been made, Figure 7, in order to calculate the behavior factor q, that is used in the design of structures to reduce the forces obtained by the linear analysis, and to take into consideration the nonlinear behavior the structure.

Satisfactory correlation between the results have been determined and value of q equal or bigger than 3 has been defined, Table 1. As pointed out by the author, this value of q=3 is specific for the observed structure and cannot be generalized for other types of structures made of Xlam.

Similarly as the three storey structure, the seven storey structure also showed no severe damages after the tests performed and confirmed the conclusion that cross laminated timber structures are exceptionally good choice for seismic active regions [4].

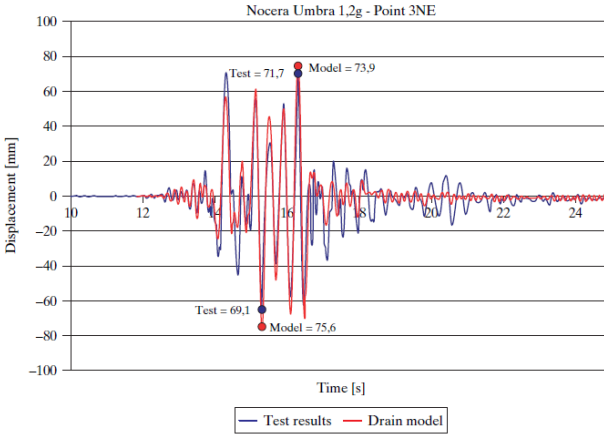


Figure 7. Comparison of the experimental and DRAIN-3DX result

Table 1. Calculated value of q

Earthquake	$PGA_{u,eff}$ (g)	Calculated q factor
Kobe	1,15	3,28
El Centro	1,20	3,43
Nocera Umbra	1,60	4,57
Northridge	0,88	2,51
Joshua	1,09	3,11
Loma Prieta	1,05	3,00
Mexico City	1,23	3,51
Kocaeli	1,43	4,09

As an addition to this wide-ranging research, supplementary cyclic tests of Xlam wall panel elements with different configurations and different type of connections have been conducted. Special subroutine for more realistic representation of the cyclic behaviour of connections have been developed and incorporated in ABAQUS software package. Results obtained from the analysis showed good correlation with the experimental results, Figure 8, proving once again that the disposition and the type of the connections used define the behaviour of this type of structural system. [6].

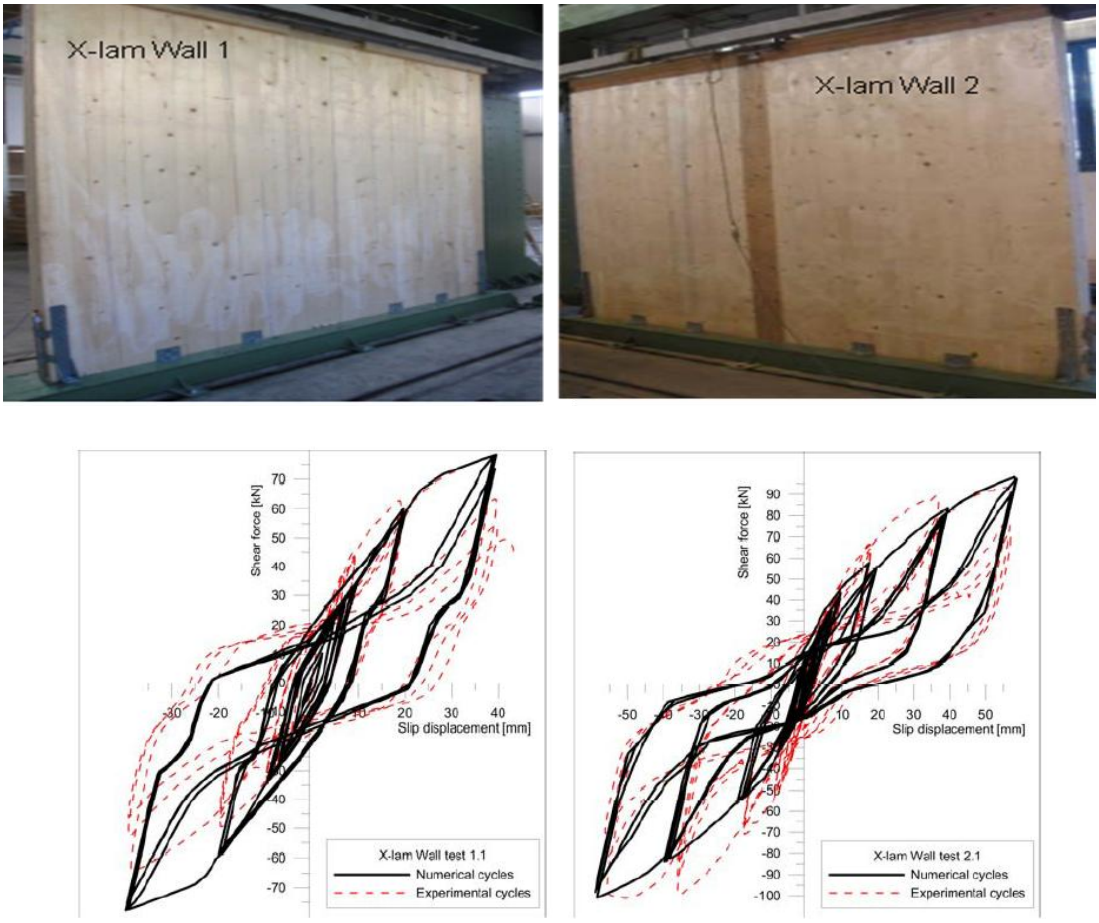


Figure 8. Cyclic tests on XLAM panels: Comparison of experimental and numerical results

Research Work in UL FGG and IZIIS SKOPJE

Parallel with prof. Ceccoti investigations, in the laboratory of the Faculty of Geodesy and Civil Engineering in Ljubljana, Slovenia, thorough research program has been realized in order to determine the seismic behaviour of massive wooden wall panel elements. XLam panels, 244/272/9.4cm and 320/272/9.4 cm, with and without openings were subjected under monotonic and cyclic horizontal load, with various boundary conditions and different constant vertical load. Testing was performed using the specially constructed test set-up for this purpose at the UL FGG laboratory, Figure 9. The main goal was to define the influence of the boundary conditions, openings, magnitude of vertical load, as well as the anchorage system on the shear capacity of XLam panel wall elements [7],[8].

Figure 9. Quasi-static tests in the Laboratory of ULFGG

The next step of this investigation program included shaking table tests on full-scale Xlam models, preformed at the Dynamic Testing Laboratory in IZIIS, Skopje. Model 1 consisted of one unit wall elements 244/272/9.4cm and Model 2 consisted of two wall elements assembled by screwing together two basic units of 122 cm length, Figure 10. Series of harmonic and seismic tests were performed and based on the obtained results reliable analytical model was developed using FELISA/3M software package. Special constitutive relationships describing the behaviour of the anchors and the contact zones were developed and incorporated into the FELISA/3M. Reasonable correlation between experimental and FELISA/3M results was achieved. The verified numerical models were used for determination of behaviour factor q, obtaining at the end value of q between 2 and 4 [9].

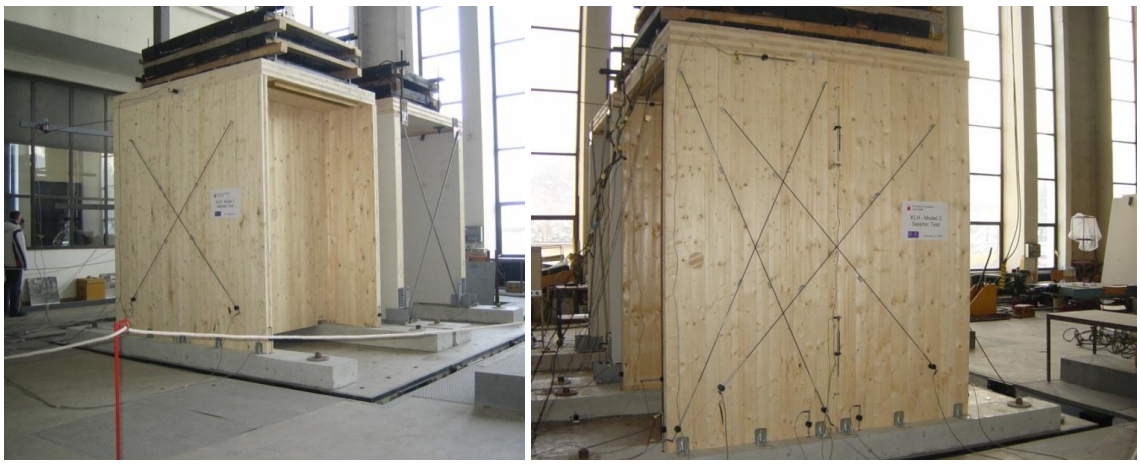


Figure 10**.** Shaking table tests in IZIIS, Skopje

Research Work in KIT

In the Karlsruhe Institute of technology (KIT), comparison between XLAM and traditional timber structures has been made via experimental and analytical investigations. Using the computer program DRAIN-2DX and based on the obtained experimental results nonlinear analytical model has been developed and behavior factor q has been calculated with average value of 4.7 for all completed tests, [10].

Research Work in Canada and USA

In the last years XLAM structures take also construction markets in USA and Canada. Several investigations related with their earthquake resistance have been undertaken by FP Innovations. 32 monotonic and cyclic

tests on XLAM panels with 12 different configurations and 4 type of connections [11]. The results from these experimental tests have been used for performance-based seismic design (PBSD) of 10 storey XLam building and definition of corresponding behaviour factor q or so called force modification factor R as stated in NBCC (National Building Codes of Canada).

Innovative Solutions with XLam

As a result of the growing interest in XLam structural system, investigations have also been focused towards upgrading of existing structures, and in that case light structural systems are especially desirable as with their low weight have almost no influence on the seismic forces within the existing structure. These points have been confirmed by the results from the dynamic tests conducted on three different constructions (two RC frames and RC frame with masonry infill) upgraded with four different construction systems: cross-laminated timber, light timber frame, steel frame and glass-timber construction, realized by the C.B.D d.o.o team from Slovenia, Figure 11 and 12, [12]. Achieved results showed that the natural periods of vibration of the RC frame structure with different upgrading depend more on the mass of the upgrade rather than its stiffness.

Figure 11. RC frame with upgrade: steel construction. Xlam construction, light timber frame and glass-timber construction

Figure 12. RC frame with masonry infill and upgrade: steel construction. Xlam construction, light timber frame and glass-timber construction

In case of upgrading stiffer structure as it is RC frame with masonry infill, change of the period is mainly influenced by the stiffness of the upgrade rather than its mass.

Another field of research and development are very high buildings made of prefabricated timber elements with combination of other materials such as concrete that is wood-concrete skyscrapers. The highest XLam building, built so far is a nine storey building Murray Grove in London, where even the elevator shafts are made of XLam. Wood-concrete skyscrapers concept uses outriggers at certain intervals, integrated tension cables and XLam structural elements in the facades, Figure 13, [13].

The construction is based on a core that joints together elevators, stairs etc. and provides proper resistance mainly against shear and to some extent against bending. Undertaken analysis showed that the construction principle is feasible and that crucial part of XLam skyscrapers design are connections between timber building and outriggers and core.

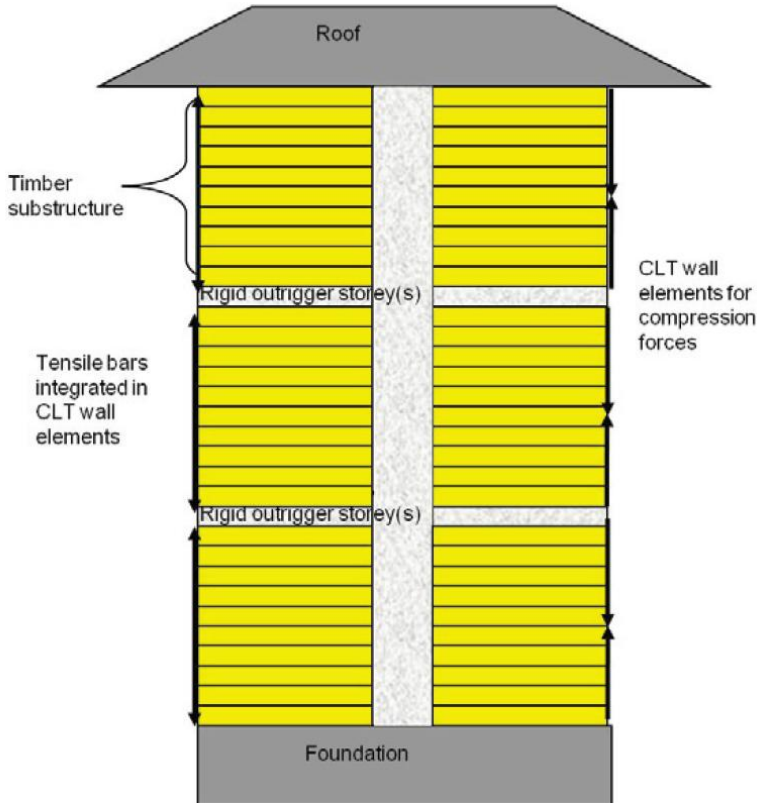


Figure 13. Wood-concrete skyscrapers concept

Furthermore, application of translational tuned mass damper on a multi-storey cross laminated timber building was analyzed, Figure 14, [14]. Numerical study was performed on a seven-storey XLam building (tested on a shaking table in NIED Miki, Japan, within the SOFIE project) proposing new approach to the design of TMDs used to reduce accelerations and control displacements in high-rise timber buildings. The results obtained show that well-designed TMDs can successfully contribute towards protection of buildings against destroyable earthquakes' effects.

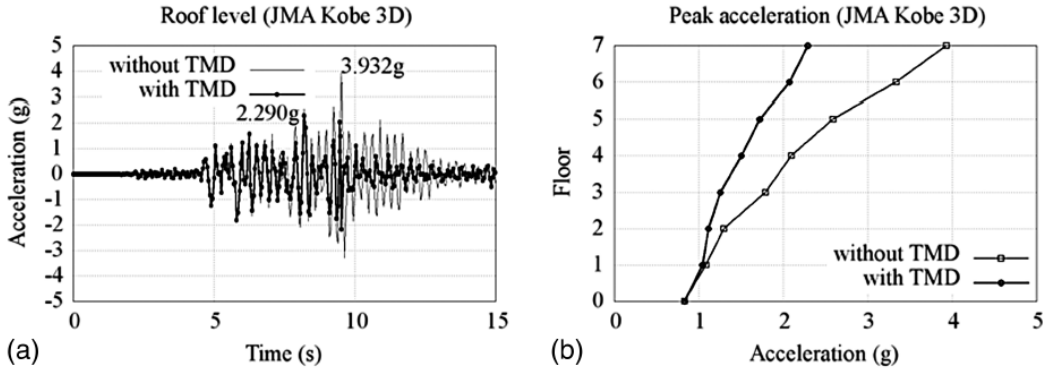


Figure 14. a) Time histories of accelerations, X direction, 6th floor; b) distribution of peak accelerations at each floor of 7-store XLam building with and without TMD under high-intensity earthquake record

CONCLUSION

Cross laminated timber is original relatively new wood based product, Cross laminated timber structures are significantly earthquake resistant, but their dynamic response completely depends on the anchors behaviour under cyclic loads. The connections are critical part in these type of structures and all nonlinearity and energy dissipation of the whole systems comes from there. Value of behaviour factor q=2 as stated in Eurocode 8 does not corresponds to the real energy dissipation capacity that XLam systems have. Results from the conducted investigations offer value of 3 or greater then 3 as more representative. Studies also show that high load-bearing capacity of XLam makes it suitable for high-rise buildings and application of tuned mass dampers in mitigation of the earthquake effects is encouraged making XLam structures advantageous alternative and serious competition to other types of structures in construction industry.

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Transmet, founded in 1992, has grown to become a leader in the Macedonian and regional construction sectors. With around 200 employees, the latest equipment and technologies, and an outstanding reputation for on-time, high-quality services, Transmet meets the highest international standards.

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The company has grown particularly rapidly over the past four years thanks to its participation in larger, more challenging projects. Sead Kočan points out, "We have been achieving double-digit growth year after year and now we are ranked among Macedonia's top five construction companies and among the country's top 50 most successful companies. Our vision is to sustain our place in this market but even more, to be the leader in international work standards implementation and those recognised as continuously re-investing our profits to better serve our clients.

Transmet's core business is infrastructure construction, but the company is also active in the mining sector through surface mining to detect mineral resources, a field in which it is Macedonia's leader. In the construction sector, Transmet has completed a wide range of above-ground and underground construction, including both infrastructure and energy projects.

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Sead Kočan, General Manager

Ideal local partner

Sead Kočan welcomes the chance to establish partnerships with international companies and investors entering the Macedonia market. He says, "Tenders are offered here every day, from building hydropower and wind plants to highway concessions, free zones and more for investors to come and invest their capital. Those investors who want to expand their influence in the region, have good references, experience and most advanced technology, are welcome to exchange experiences and knowledge with our companies. This can benefit us all. As a local partner, Transmet will provide a range of support, from consulting and market research to obtaining permits and project management. "

Transmet has achieved ISO certification for quality management and is implementing ISO 14000 environmental standards and ISO 18001 safety standards. It is also expanding into service stations and the production of construction materials. Sead Kočan says, we aim to line up €100 million in projects over the next couple of years and we welcome partnerships.

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RESPONSE BASED MODAL IDENTIFICATION OF A REAL STEEL CHIMNEY

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Measured structural responses are commonly employed via operational modal analysis (OMA) methods in order to assess the modal parameters of the monitored system. Vibration data recorded within a semi-continuous Structural Health Monitoring (SHM) campaign installed on a real steel chimney structure is in the focus of this study. The dynamics of the actual structure is studied under real loading scenarios for two specific load cases. By the utilization of a combined response based approach the natural frequencies of the steel chimney structure are successfully identified.

Keywords: real structure, vibration response, structural monitoring, modal identification

1. INTRODUCTION

The attractive potentials in exploiting measured structural responses in operational conditions under natural excitation, introduced in the early 1990's, marked the beginning of the OMA era. Indeed, the advantages in identifying the actual structural dynamics by conducting fast and relatively cheap experiments have instantly attracted the research and industry communities [1].

The following vast progress in the development of various assessment tools suitable for accurate and reliable modal identification of monitored systems, as well as the parallel technological advancements, have tunneled the focus towards development of more holistic approaches suitable for automated SHM damage detection and life-time estimation strategies on existing structures [2].

Unlike the controlled laboratory environment, implementing SHM strategies on real civil engineering structures introduces many uncertainties, uniquely linked to the particular system under study. This dismisses the possibility for straightforward utilization of existing structural identification tools. In this context it becomes an imperative to test the

performance of various OMA methods on real structures.

Vibration response data was continuously collected for the period of two months in the framework of an implemented monitoring system on a steel chimney structure. By exploring the performance of the RD-EFDD (Random Decrement – Enhanced Frequency Domain Decomposition) method the focus of this study is: 1) extraction of the natural frequencies of the monitored structure and 2) assessment of structural behaviour for selected loading scenarios, specific for the actual structure.

2. RD-EFDD ANALYSIS APPROACH

The RD-EFDD method is a merged approach which enables estimation of the Averaged Normalised Power Spectral Densities (ANPSD) of response variables with reduced noise and reduced effects of leakage, compared to the commonly applied frequency domain tool (EFDD). The concept was verified for the case of a scaled laboratory model of a frame structure [3].

Under the assumption that assembled structural responses are realization of a zero mean stationary Gaussian stochastic process, the RD functions are proven to be proportional to the correlation functions of the responses and/or to their derivatives in relation to time [4-6], thus enabling the employment of RD functions within OMA framework.

The RD functions are interpreted as free vibration responses of a system subjected to random input loads. Namely, if we decompose the system's response to the three parts: i) response to an initial displacement, ii) response to an initial velocity iii) response to the random loads between initial time and time *t*, then by averaging the time segments with identical initial conditions the part related to random inputs has the tendency to diminish.

The RD technique is a time-domain technique and may be utilized with various triggering conditions applied directly to the response time-histories [6]. In this manner, time segments corresponding to the same triggering condition are extracted from the full length samples and averaged. The newly obtained signals are the RD functions. By evaluating the auto and cross RD functions (auto-triggering or cross-triggering condition) of measured response data, the RD functions matrix is constructed. Finally, the ANPSD matrix can be evaluated as the mean of the spectral matrices computed from each column or row of the RD function matrix [3].

Eventually, the EFDD method is applied in the same way as with the commonly estimated spectral densities from vibration responses [7].

3. THE APPLICATION CASE STUDY

The structure under study is an out of use steel chimney located on the premises of Ohis Factory in Skopje, Republic of Macedonia (Fig.1).

Height: 40 m

Diameter: 1.9 m

Thickness: 6-10 mm

Material: Steel S235

Construction year: 1970

Figure 1. Monitored steel chimney

In order to obtain a comprehensive dynamical model of the chimney structure, a SHM campaign was undertaken in the period 14/12/2013 to 14/02/2014. An installed monitoring system for the complete timeframe of two months continuously measured: structural vibration responses, environmental parameters (wind velocity and ambient temperature), as well as ground vibrations nearby the structure (Fig.2).

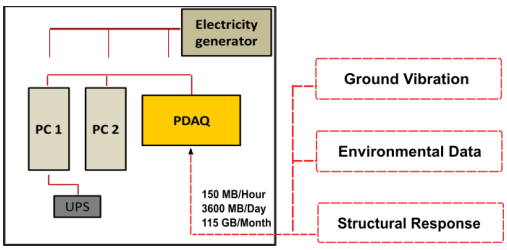

Figure 2. Schematic overview of the SHM system

One-hour long acceleration time histories are recorded by five tri-axial accelerometers for ambient vibration (Digitexx D110-T sensor) placed along the structure's height, with the sampling frequency of 200 Hz. The placement and positioning of the sensors, as well as the

measured response directions are presented in Fig. 3. As a preliminary step and for the purpose of comparison only, a Finite Element Model (FEM) (beam and shell) of the structure was developed (Fig. 4).

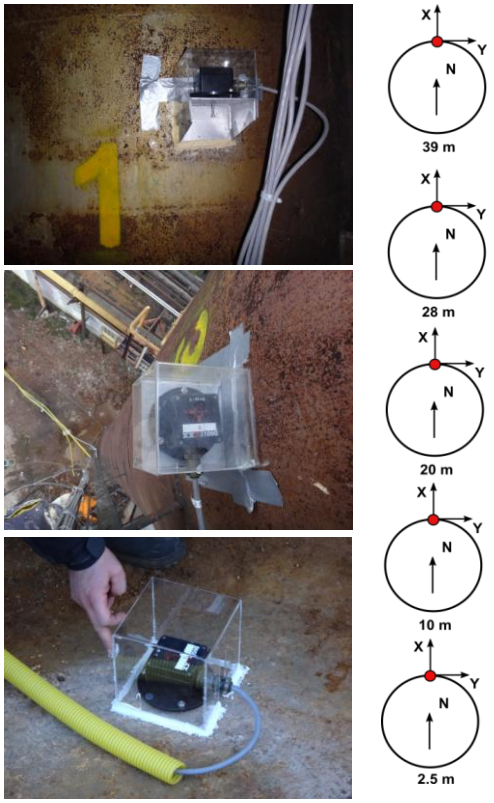

Figure 3. Accelerometers installation & position along the structure height and cross sections

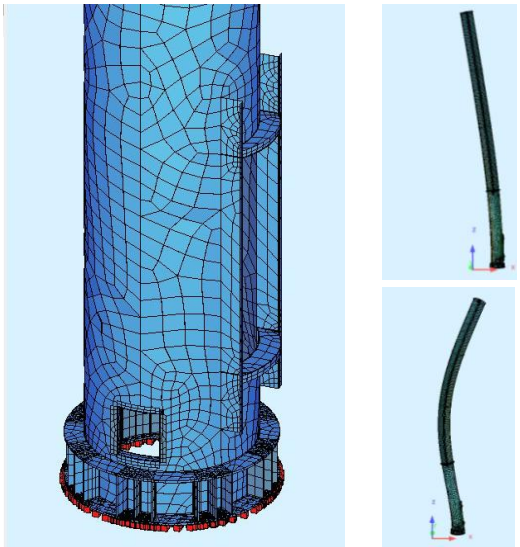

Figure 4. Shell FEM of the structure

The previously introduced response-based method is herein applied for an initial structural behaviour assessment related to two identified loading scenarios: i) recurring train induced vibrations (TIVs) from a nearby railway, and ii) wind induced vibrations (WIVs) for a time frame corresponding to the maximal value of recorded wind velocity within the two months period of monitoring. In Fig. 5 the 24-hour (25.01.2014) measured wind velocity is presented, including the period of the highest recorded value of 21 m/s.

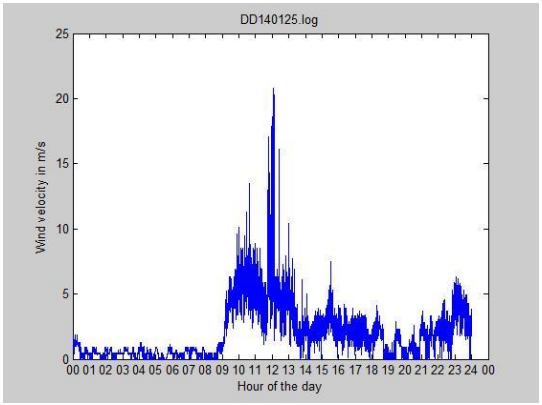

Figure 5. Time-history of the wind velocity containing the maximum measured value

Before further utilization, the acceleration time histories for both cases were low-pass filtered and down-sampled to 50 Hz (cut-off frequency at 17 Hz).

The spectrograms (Short Time Fourier Transform; Hamming data window; NFFT = 1024; overlap 98%) of measured vibration response corresponding to the highest sensor (X direction in Fig.3) clearly expose the specifics of each case (Fig. 6 and Fig. 7). The train induced amplifications in the acceleration time history are as well clearly visible for the selected 30-min data sample (Fig. 6).

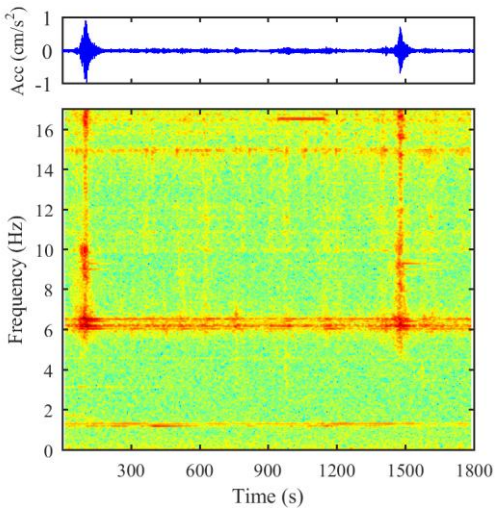

Figure 6. Case of TIVs- top sensor Xdir: (up) Acceleration time-history, (bottom) Spectrogram

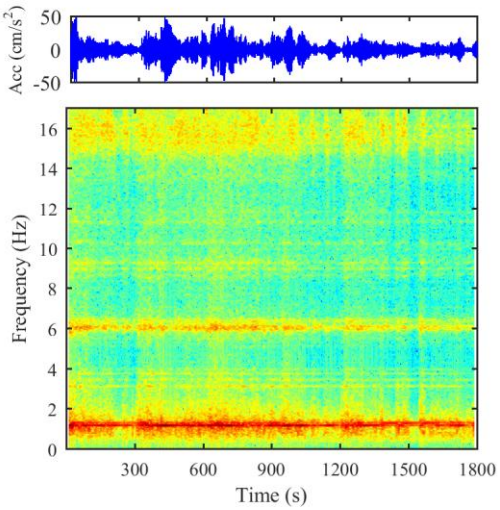

Figure 7. Case of WIVs - top sensor Xdir: (up) Acceleration time-history, (bottom) Spectrogram

An interesting observation is the identical frequency "signature" (excluding the structural frequencies) revealed by further inspection of the time-frequency plots of ground accelerations (accelerometer placed nearby structure) (Fig. 8).

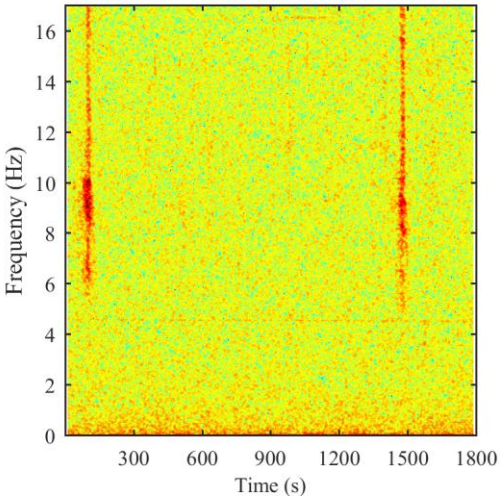

Figure 8. Case of TIV: Spectrogram of the ground acceleration record

The RD-EFDD method is now utilized for obtaining the structural modal parameters of the chimney structure. The triggering level crossing condition is selected as a multiple of the standard deviation (std) of the measured accelerations: 1std (TIVs) and 4.5std (WIVs). The time segments length necessary for complete decay of the RD functions is 1500 data points (dp) for the TIVs case, and 6000 dp for the WIVs case, which corresponds to 30 sec and 120 sec long signals, respectively.

In Fig. 9 and Fig. 10 the ANPSD plots for the two selected load scenarios are presented. The TIVs introduce amplification in the higher frequency range (as clearly indicated by the spectrograms in Fig. 5 and Fig. 6). On the other hand the WIVs accentuate the lower range of frequencies, however as well enabling a clear identification of the first two natural frequencies of the structure.

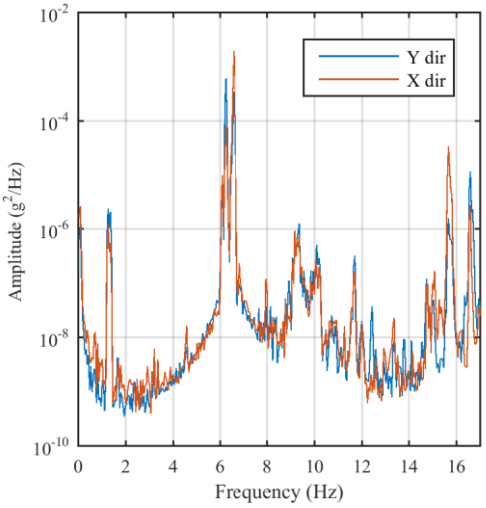

Figure 9. Case of TIV: ANPSD from RD-EFDD method (triggering: 1 std, length: 1500 dp)

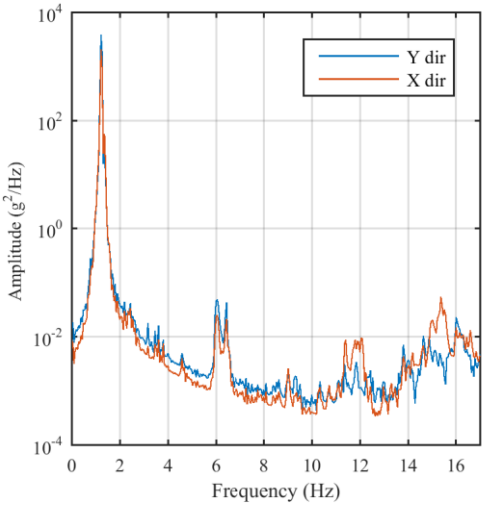

Figure 10. Case of WIV: ANPSD from RD-EFDD method (triggering: 4.5 std, length: 6000 dp)

In Tab. 1 the comparison of the estimated frequencies from the FEM of the structure and data-based estimations via the RD-EFDD technique are presented. Only with the purpose of validation, results from the commonly applied stochastic subspace identification method (SSI-COV) are included as well. For the sake of brevity the details of

the method will not be reported herein (for an overview the reader is referred to [8]).

The RD-EFDD based mode shapes corresponding to the identified natural frequencies are presented in Fig. 11.

Table 1. Comparison - list of the first two estimated natural frequencies of the chimney structure (in Hz)

f *[Hz]*	**FEM (shell)/ (beam)**	**WIV EFDD-RDT**	**WIV SSI-COV**	**TIV EFDD-RDT**	**TIV SSI-COV**
$f_{[1\text{-}x]}$	1.32/1.38	1.18	1.17	1.19	1.19
$f_{[1\text{-}y]}$	1.39/1.38	/	1.25	/	/
$f_{[2\text{-}x]}$	6.54/6.95	6.00	6.04	6.20	6.18
$f_{[2\text{-}y]}$	7.35/6.95	6.39	6.39	6.54	6.55

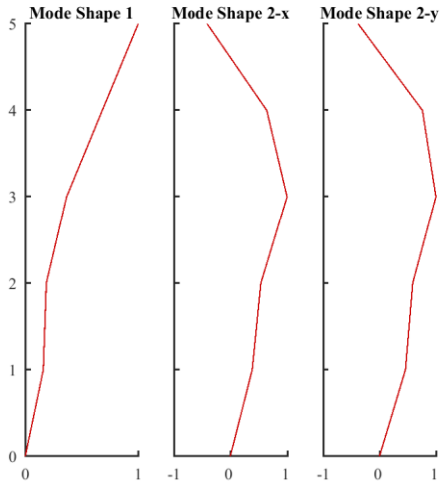

Figure 11. RD-EFDD estimated mode shapes for the identified structural frequencies

CONCLUSIONS

Recorded response data of a real steel chimney structure was successfully utilized to identify the first two natural frequencies of the vibrating system.

The RD-EFDD method shows good agreement with results from the simulated FEM analysis and SSI-COV estimates. However, the approach was not capable of identifying the closed spaced first natural frequencies of the structure in both load cases.

Inspection of the spectrograms of measured accelerations for the two different load cases (train and wind induced vibrations) revealed the specifics of the dynamic response of the system.

Following research is directed towards assessing modal damping parameters, close spaced modes, as well as identifying higher structural frequencies.

Acknowledgements

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COMPARISON OF CONTINUOUS COMPOSITE BEAMS BEHAVIOUR

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Composite steel and concrete structures are best solution for decks on steel structures. The composite steel and concrete continuous beams have nonlinear behaviour even on the lower values of the ultimate plastic moment resistance. For the needs of larger volume of relevant data, experimental trials are conducted for testing the different behaviour of the composite continuous beams. Therefore, the experimental testing was conducted on two identical composite continuous beams, with various principles of improvements of the bearing capacity, especially at the middle support. For obtaining greater value of the bearing capacity of the beams, the first model is reinforced with two layers of mesh reinforcement at the middle support, where the second model, with the same reinforcement, is additionally pre-stressed by controlled imposed deformations. In this paper the results of the behaviour of the mentioned beams are presented, compared with each other.

Keywords: composite structures, composite slabs, steel structures

1. CHARACTERISTICS OF THE MATERIALS

Materials used for composing the continuous composite beams are in accordance with the Euronorms. The characteristics of all materials were previously tested, where the yield strength, the tensile strength and the maximum elongation of a representative sample of the steel elements are determined. For the main beam is used S275JR, with tested yield strength f_y=275N/mm^2, and tested tensile strength f_u=424N/mm^2, with k=f_u/f_y=1.54 and ε_u=18.9%, according to EN1993. The material used for the shear connectors is S235J2+C450, with f_y=502N/mm^2, and f_u=552N/mm^2, where k=f_u/f_y=1.10 and δ_5=18.5%, according to EN13918. The material used for the steel sheeting deck is S550GD Z275, with f_y=675N/mm^2, and f_u=770N/mm^2, where k=f_u/f_y=1.14 and δ=24.6%, according to EN10147. The material for the reinforcement is class B, according to

EN1992, with f_{yk}=597N/mm^2, and f_{uk}=662N/mm^2, where $k=f_{uk}/f_{yk}$=1.11 and ε_{uk}=9.9%.

The material for the concrete is predicted for the strength class of C25/30 according to EN1993. Testing the characteristics of the casting for the first model "ГН1", the compression strength of cube is $f_{ck,cube}$=36.64MPa, the compression strength of cylinder is f_{ck}=29.51MPa, the tensile strength is f_{cm}=2.95MPa, and the modulus of elasticity is E_{cm}=31.97GPa. The shrinkage at the time of the loading of the first model, at the 29th day of casting, is measured 0.255%, where the calculated value of the shrinkage in accordance with EN1992 is 0.230%. Testing the characteristics of the casting for the second model "ГН2", the values are $f_{ck,cube}$=32.03MPa,f_{ck}=31.65MPa, f_{cm}=2.97MPa, and E_{cm}=31.65GPa. The shrinkage at the time of the loading the second model, at the 32nd day of casting, is measured 0.241%, where the calculated value of the shrinkage in accordance with EN1992 is 0.273%.

The results from the testing the characteristics of the materials meet the requirements given in EN1992 for the concrete, and EN1993 for the steel.

2. TESTING THE CAPACITY OF THE SHEAR CONNECTORS

The behaviour of the shear connectors was also tested, and is part of this complex experimental and theoretical research of the continuous composite (steel and concrete) beams. The testing is conducted in accordance with EN1994, on three separate modified models.

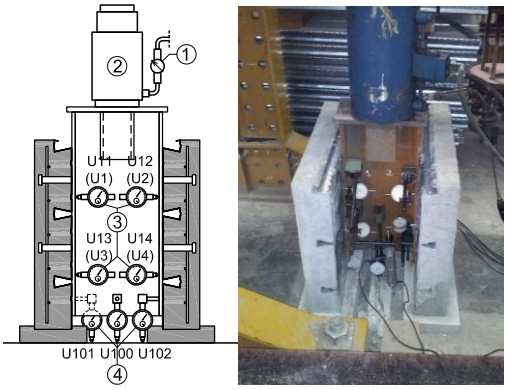

Figure 1. Testing model with applied measuring instruments

Where: 1) dynamometer for load measuring; 2) 100 ton press for load application; 3) dial (U11 to U14) and electronic (U1 to U4) comparators for transverse separation between the steel beam and the concrete slab; 4) electronic comparators (U100 to U102) for longitudinal slip.

The electronic equipment is connected to HBM Quantum data acquisition system amplifier, directly connected to personal computer. The measuring is carried out through the testing in real time.

For this testing it's used the same cross section of the steel beam, the same materials, the same strength class of concrete and reinforcement as the main testing models. The materials meet the requirements according to EN1994.

The objective of this testing is obtaining information for the achievement of full plastic capacity of the designed cross section, and obtaining the "P-δ" diagrams for the theoretical research, which is scope for another paper.

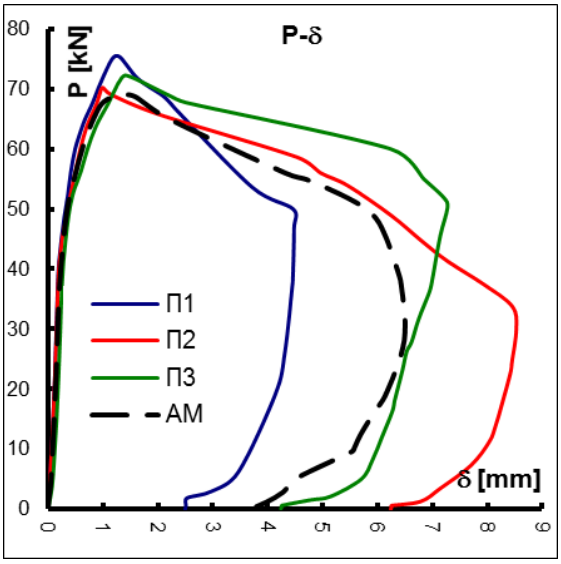

Figure 2. "P-δ" diagrams of the tested models

From the testing, among other things, it is concluded that the shear connectors have the required capacity for development of full plastic moment resistance of the designed cross section.

3. TESTING MODELS, MEASURING AND LOAD APPLICATION EQUIPMENT

For the purpose of the testing, two identical models were created, first model "ГН1" as composite (steel and concrete) continuous

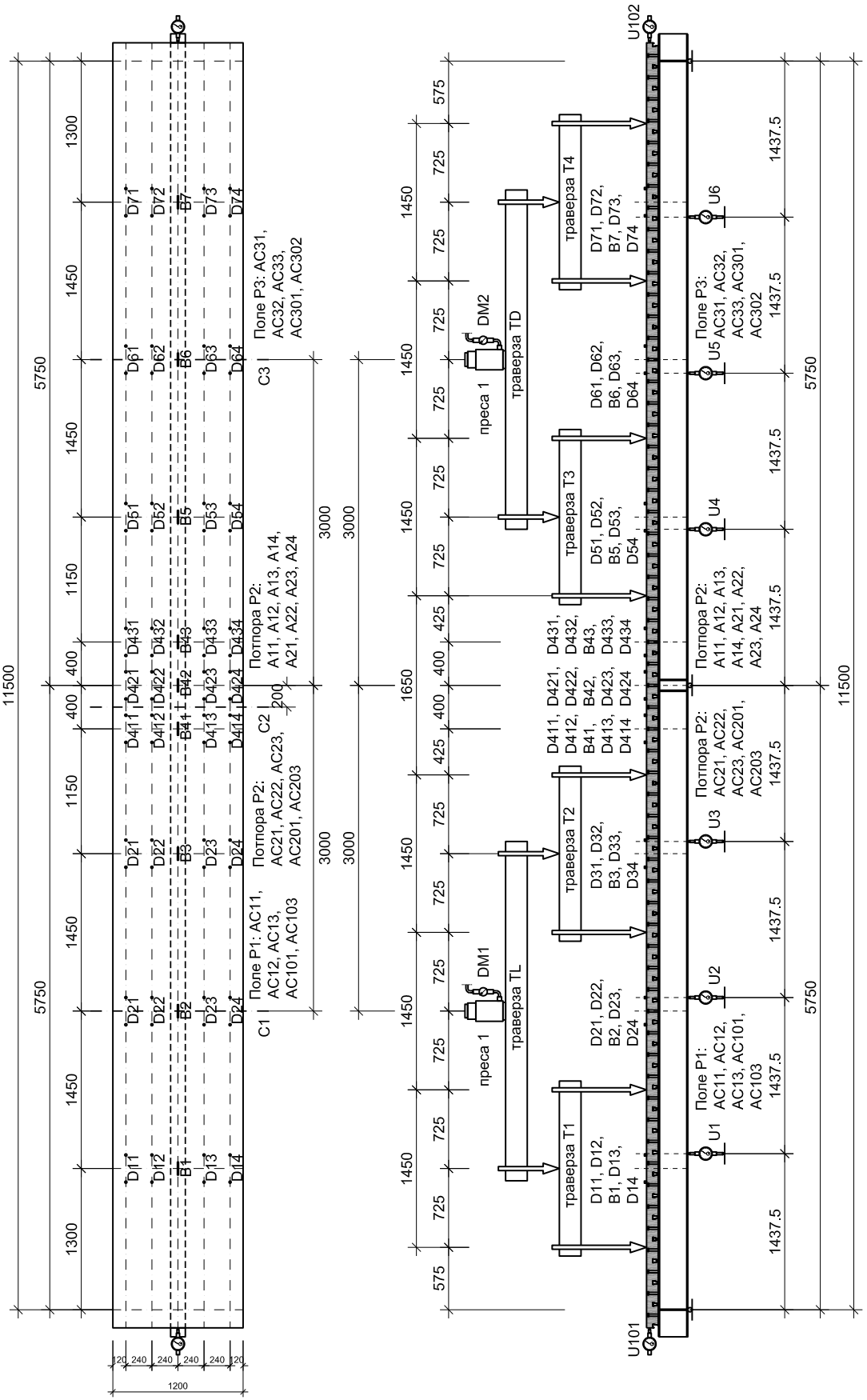

Figure 3.Testing model with measuring and load application equipment

beam, and second model "ГН2" as composite continuous beam pre-stressed by controlled imposed deformations. The beams are with "T" beam cross section made from IPE200 steel beam (1) and composite deck with steel sheeting Bondeck 600, with thickness of 1.0mm, and depth of 52mm (2). The full depth of the composite deck is 52+58=110mm, reinforced with Q273 (Ø6/100mm) (4). At the support there are two layers of the same reinforcement, for improving the bearing capacity of the concrete. For shear connectors Nelson headed studs (3) are used, with diameter of 19mm and height of 100mm, as shown in figure 4. The static system is continuous beam, with two spans of 5750mm.

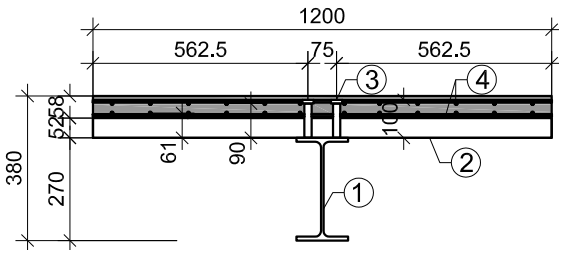

Figure 4. Cross section of the testing models

In figure 3 the loading and measuring equipment are shown. The force is introduced to the beam through two layers of transmission beams, where from two loading points, the load is distributed on eight points longitudinal to the beam, with that simulating uniform load. The load is applied through two 100 tons presses, for each lading case the force is measured with the usage of the dynamometers.

The displacements are measured with electronic comparators U1 to U6, and with dial comparators U101 and U102 the slip between the steel beam and concrete slab is measured. The strains on the top face of the concrete slab are measured with dial deformeter with main base of 250mm. The measuring points for the deformeter are D11 to D74. In the middle of the width of the concrete slab, strain gauges with main base of 120mm are used, B1 to B7. Strain gauges, with main base of 5mm, are used for measuring the strains in the steel section, AC11 to AC303, and also for measuring the strains in the reinforcement, A11 to A24.

At the end of each step, the data from the strain gauges was obtained through HBM Universal amplifier, and the data from the dynamometers and the electronic comparators was obtained from HBM Quantum data acquisition system, in real time.

The difference between the models is that the second model is pre-stressed with controlled imposed deformations. According to the analytical research, 18mm of imposed deformations is enough for the most optimal and effective improvement of the bearing capacity of the cross section at the middle support. The steel beam is constructed with elevation at the middle support of 18mm, then the steel sheeting, the welding of the shear connectors and casting of the concrete were made. After the curing of the concrete, the lowering of the middle support was conducted, osculating the behaviour and measuring the displacements and strains. With pre-stressing by the imposed deformation a compression force on the concrete slab is induced at the middle support.

The first model "ГН1" is loaded with cycled load up to 40kN in dynamometer (10kN at the loading point in contact to the testing model), then the load is applied in subsequent increments until failure occurs (up to 450kN). After every step of loading, values from the measuring equipment (deformeter and dial comparators) are obtained. From the electronic dynamometers, comparators and strain gauges, the measurements are obtained in real time throughout the testing. The second model "ГН2" is loaded similar as the first model, but the load when the failure occurs is 600kN.

4. RESULTS FROM THE TESTING

The obtained data from the testing of the composite beams are adequately processed in tables, drawings and diagrams with measured strains, deflections and stresses. The appearance and the width of the cracking of the concrete at each loading step were also obtained.

The results from the testing are analyzed and compared with the procedures given in EN 1994 and with realistic complex 3D model created in the structural software for analysis and design SAP2000. In this paper, only the results from the testing are provided and compared.

4.1 FIRST MODEL "ГН1"

The first crack in the concrete is at the support with load step of 60kN in dynamometer (15kN at the loading point in contact to the testing model), producing moment around 36.37kNm. The most exposed fiber of the steel cross section at the support reached the yield

strength in loading steps between 225kN and 250kN in dynamometer. The first full yielding of the steel cross section, where the ultimate limit state is reached, is at the support at loading step of 450kN, producing moment of 234.12kNm. The ultimate limit state of the composite beam at the span is reached at the loading step of 512kN in dynamometer (128kN at the loading point in contact to the testing model).

The elastic deflection of the first tested model is 12.14mm, where the deflection with plastic behaviour of the beam is 24.41mm at the loading step of 450kN in dynamometer.

4.1 SECOND MODEL "ΓΗ2"

The first crack in the concrete of the pre-stressed continuous composite beam is at the loading step of 140kN in dynamometer. The yield of the most exposed fiber of the steel cross section is at the loading step of 375kN at the spans of the beam. The full yielding of the steel cross section at the support occurs at the loading step of 550kN, producing moment of 244.13kN, where the ultimate limit state of the cross section at the spans is at the loading step of 618kN (154.5kN at the loading point in contact to the testing model), producing moment of 297.30kNm.

The elastic deflection of the tested model is 13.04mm, where at the step of loading of 450kN in dynamometer is 20.12mm, and 51.32mm for the ultimate limit state of the beam.

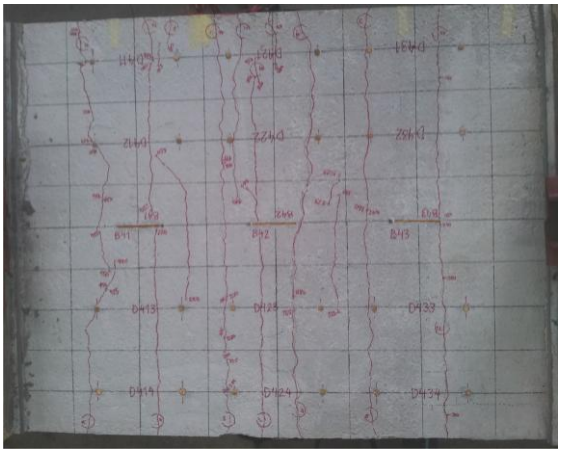

Figure 5. Cracks in the concrete slab at the support, final loading step of "ΓΗ2"

5. COMPARISON OF THE RESULTS

From the obtained data from the testing of the two models, difference in the behaviour of the models is observed.

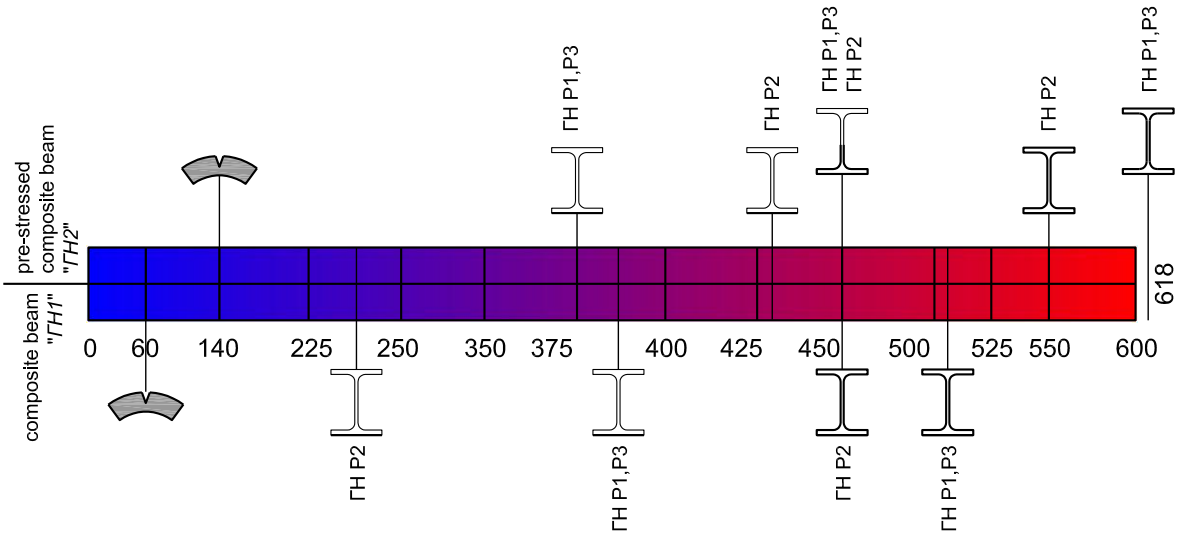

Figure 5. Display of the compared results of the tested models

It is obvious that the pre-stressed composite continuous beam "ΓΗ2" has more advantages from "ΓΗ1", in the field of the serviceability and ultimate limit state.

The cracking of the concrete is delayed for 80kN in dynamometer, which is a great advantage for the serviceability limit state of the structure and a great advantage for the elastic behaviour of the composite beam, which results with loading difference of 15.4kN/m'.

The full yielding at the support of "ΓΗ1" is at the loading step of 450kN in dynamometer, where for "ΓΗ2" is at 550kN, with difference of +22.2% or with loading difference of 19.3kN/m'. The ultimate limit state of "ΓΗ1" is at the loading step of 512kN in dynamometer, where the same for "ΓΗ2" is at loading step of

618kN, with difference in +20.7% or with loading difference of 20.5kN/m'.

There are advantages of "ГН2" in the deflections compared to "ГН1". At the step of loading of 450kN, the deflection of "ГН1" is 24.41mm, where for "ГН1" at the same step of loading is 20.12mm, with difference of -17.6%.

6. CONCLUSIONS

Having the results from the testing of the two composite continuous beams, and with the analysis of the results, some conclusions can be drawn.

It is obvious that the behaviour of the tested beams are completely nonlinear, especially after the occurring of the first cracks in the concrete slab at the support.

One of the advantage is greater serviceability limit state of "ГН2" compared to "ГН1", where the pre-stressed composite beam can be load with additional 15.4kN/m' until the first crack in the concrete at the middle support occurs.

In the elastic behaviour, the advantages are 2.33 times higher bearing capacity of "ГН2" compared to "ГН1". There is 1.88 times higher bearing capacity after the occurring of the cracks until the yield strength of the most exposed fiber is reached.

The pre-stressed composite beam "ГН2" has 20.7% higher ultimate limit state than "ГН1", which means that it can be loaded with additional 20.5kN/m' until the bearing capacity is reached. The ultimate load, when the ultimate limit state of the beam is reached, for "ГН1" is 98.94kN/m', where the ultimate load for "ГН2" is 119.24kN (+20.7%).

It can be concluded that with pre-stressing the composite beam by controlled imposed deformations by 18mm, there are many advantages in the behaviour of the beam compared to ordinary composite beam, and that the advantages are in the better serviceability and ultimate limit state of the beams.

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ISO STANDARDIZATION IN GEODETIC COMPANIES

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In this paper, principle and strategy for the integration of basic IMS (ISO 9001 Quality Management Systems - QMS, ISO 14001 Environmental Management Systems - EMS, and OHSAS 18001 Occupational Health & Safety Assessment Series) are elaborated in process management of geodetic companies. IMS politics should be incorporated into overall business strategy of the geodetic company, in order for permanent improving the work quality of the company, protection and improvement of the environment, maintenance and continuous improvement of health protection and safety at work. Standards for quality management provide a wide range of benefits of economic, technological and social aspect in the organization - internal and external benefits in the overall business environment.

Keywords: International organization for standardization (ISO), integrated management systems (IMS).

1. INTRODUCTION

The globalization and world modern trends in economy, the innovations and application of modern information technology, as well as more frequent continuous changes in organizations, have a strong impact on global economic development. The tendencies of company managers are to increase their organizations in global business environment, to be competitive, recognizable, and in some segments unique in order to survive in present dynamic world of business. These modern economic trends didn’t blow past the geodetic profession at all. The impact is directed to revolutionary change of applying of classical methods for acquisition, processing and distribution of spatial data.

The development of modern surveying organizations in general involves integration of management by providing a qualitative final product and service to end users.

Implementing systems of quality management in modern geodetic organizations should be a strategic decision that will ensure the monitoring of changes in the environment, realizing the needs and achievement of objectives, quality of service and uniformity of processes in the organization.

2. STANDARDS

In accordance with the definition of the ISO organization: *The standard is a document that provides conditions, specifications, guidelines or characteristics that can be used consistently to ensure that the materials, products, processes and services are eligible for their purpose*.

The international ISO standards in process of leading business are strategic tools that provide cost reduction in working processes, reduction of waste and errors in working process, increased productivity, safety and protection at work as prerequisites for entry and penetration of new markets [1]. The standards provide also protection of end users and product consumers, ensuring that the certified products or services are in accordance with minimal standards set on national level.

The ISO organization published more than 20493 international standards till 2014, covering almost all sectors and areas of industry, technology, transport, environment etc.

Besides the big number of international standards issued by ISO organization that are applicable to each area or sector separately, in global level the largest application with big popularity are following standards:

- ISO 9001 Quality management;
- ISO 14001 Environmental management;
- ISO 3166 Country codes;
- ISO 26000 Social responsibility;
- ISO 50001 Energy management;
- ISO Risk management;
- ISO 22000 Food safety management;
- ISO 27001 Information security management;
- ISO 45001 Occupational health and safety and etc.

The first two internationally recognized standards (ISO 9001 Quality Management and ISO 14001 Environmental management), are among the most popular standards of organization. That is because of development of global market and business with products and services and more increasingly need for environmental protection, whose degradation is result of global industry growing.

2.1 QUALITY MANAGEMENT SYSTEM ISO 9001 QMS

ISO 9001 standard represent a framework for implementation of a system for managing the quality of products and services within the organization. This standard for the first time is published by ISO organization in 1987.

ISO 9001 standard with certainty is one of the standards which are implemented in a number of organizations, commercial companies, service companies and other organizations [5], [6]. It regulates the system of quality management in the organization that is confirming that the organization is continuously able to provide:

- products or services that meet the expectations of users and satisfies the requirements of the legislation,
- quality through the existing system of management and improvement of processes in the organization.

This standard is not with purpose on unification of processes across organization, because it is respecting the uniqueness of each organization. The development and implementation of standard means through the process approach to ensure quality, not only of the product or service, but quality in terms of the overall operations of the organization, customer relations, employee relations, working culture, legislation and community at all.

A special target group for this standard does not exist. It is intended and may find application in any organization which activities are aimed to continuously improve and maintain quality across all processes in the organization, regardless of its size, sector or category.

The benefits of proper implementation and application of the standard ISO 9001 (QMS) are at the area of financial operations, improvement of profitability due to the reduction of losses and costs, increasing of productivity and competitiveness continuity in the quality of products and services, conquest of new markets etc.

2.2 ENVIRONMENTAL MANAGEMENT SYSTEM ISO 14001 EMS

The ISO 14001 standard first was published in 1996. This is a worldwide standard with more than 300 000 issued certificates during the 20 years of existence.

This standard is the main standard which defines the conditions for environmental management. It is ranked on priority level in organizations which operations are having an impact on the environment [7]. The reason for this is the alignment of strategy development, which results in increased exploitation of natural resources and energy on one side, and the development of human society on the other.

ISO 14001 is in group so called "risk management" standards which aim is process and risk control of environmental pollution, with establishing of specific mechanisms for environmental protection, which would reduce risks and the number of incidents.

The basic objective of the ISO 14001 standard within the organization as a whole and the employees as individuals is through certain activities in business processes to reduce environmental impact. In this sense the basic requirements of this standard are:

- to establish politics for environment prevention;
- determining environmental aspects that are arising from the implementation of the product or service in terms of environmental impact;
- align of organization working processes with the legislation for environmental protection;
- establishing a strategy and program for the implementation of policy for the protection of the environment through the implementation of general and individual goals;
- preparedness to react in changed circumstances and incidental situations.

The benefits of implementing the system of environmental protection within an organization are: reducing negative impacts on the environment and the risk of environmental disaster with activities provided for fast and efficient handling and intervention, reducing hazardous waste and savings in consumption of energy and materials [4].

2.3 OCCUPATION HEALTH AND SAFETY ASSESSMENT SERIES OHSAS 18001

The OHSAS 18001 as well as ISO 14001 belongs to the group of "risk management" standards. It aims to define and establish requirements for management, health, protection and safety at work. It is intended for all organizations, no matter their size and activity, where in work process there is a risk to the safety and health of employees. This standard is applied in organizations aimed to continuous improvement of safety in the working environment, prevent of possible damage and reduce harmful impacts on the health of employees [8], [9].

This standard is from organization British Standards Institution - BSI established in 1999. The last update of the standard was in 2007 when was harmonized with structure of ISO 9000 and ISO 14000 standards for easy implementation and integration with organizations that already have established management systems.

The system (standard) for managing health and safety OHSAS 18001 promotes a safe and healthy working environment by providing a framework that helps organizations to:

- identifying and controlling risks of health and safety;
- reducing the potential accidents;
- legal compliance with the law regulations;
- improving the overall efficiency.

The main goal of the system is to ensure the continuous reduction of occupational risk and dangers in the workplace, with feedback in: improving efficiency with reducing the number of accidents, also protection of employees, property and equipment for increasing on reputation of organization. The integration of OHSAS standard with present systems of management (QMS, EMS etc.), had impact on increasing the performance of the overall integrated system of management in the organization.

3. MANAGEMENT SYSTEMS IN PRIVATE GEODETIC COMPANIES

Registration and functioning of private geodetic companies in Macedonia begins with realization of the project *Real Estate Cadastre and Registration*, which was financed by a loan from the World Bank in 2005. With this project, in frame of *Law for survey, cadastre and registration of real estate rights* [10], part of the public authorities of in that time the State Authority for Geodetic Works are transferred to the private geodetic companies. Those authorities are related in terms of performance of the operative field geodetic works connected with real estate cadastre.

With the adoption of the *Law for valorization* [11] and the *Law for court expertise* [12], part of geodetic companies, in addition to the core business ascertained the opportunity to expand their activities in these areas and increase its offer to potential customers.

With the expansion of the activities and services that are offered from geodetic companies, is more than necessary to take some changes in organization and structural activities from the aspect of management. Although it is about similar segments, they substantially are different in methods and procedures for preparation of final products or services such as:

- *Geodetic Elaborate*;
- *Report for valorization of real estate*;
- *Expert report and opinion.*

Though from statistical data about the number of companies registered in the Republic of Macedonia can be concluded that the number is steadily growing, but there is a small number of companies that are able besides the core business (performing surveying work in the field of real estate cadastre), to offer and performing services in the appraisal of market value and court expertise in field of real estate.

Implementation of management systems is one of the factors which aim is: improving and promoting the quality of work, continuous improvement and maintenance in environmental protection and health protection and safety at work. There are no major differences in the phases of approaches and implementation of management systems for geodetic companies compared to other companies, regardless of whether it companies which main job is production or service activities.

The first step in process of implementation of management systems is an engagement of consultant with experience, references and contacts with relevant certification companies. There are cases when integrating systems management are carried out without engagement of a consultant, but certainly it is a difficult way with errors in the entire process - from the preparation of the system documentation and its establishing, till its maintenance.

The classic approach provides in the first stage of implementation to implement the system of quality management ISO 9001 QMS (Quality Management Systems), and then to implement any else's management system. Opposite of that, modern approach and recent practical experience preferred implementation of integrated management systems - IMS (Integrated Management System). IMS is the best system that simultaneously satisfies the users of services, restructuring the company, reduce costs, increase productivity and profits as well as long-term development and effective way of the company managing.

3.1 PROCEDURES IN PRIVATE GEODETIC COMPANIES

Implementation of integrated management systems in the private geodetic companies largely represents decision of the company management. For those companies seeking to follow modern business trends and to be recognized in their professional environment with orientation to customer satisfaction, IMS is a challenge from an organizational and also from professional aspect.

The satisfaction and fulfillment of the standards envisaged by IMS are conditioned with fulfill of the procedures and records prescribed by the laws and regulations that regulate geodetic activities. It is already pointed out that some of the geodetic companies that meet the professional and personnel potential, in addition to the basic geodetic activities, expanded its activities and the provision of services in the field of valorization on property market value and Court expertise related to solving legal property disputes. This means that it is a large number of real estate data, different types of geodetic elaborates depending on their purpose, expert reports and expert opinion that require good organizational structure, access and operation of the database.

3.1.1 Procedure - Real estate cadastre and engineering geodesy

The procedure registered as "Real estate cadastre and engineering geodesy," represents basic procedure if we take into account the main objective and activity of the majority of registered geodetic companies. Their work they base on preparation of geodetic elaborate in the procedures for solving and registering property rights in real estate cadastre. In the country there is a smaller part of geodetic companies that are operating in the segment of engineering geodesy. The reason for this is lack of personnel, technical and professional equipping, also lack of adequate authorizations with accordance to *Law for*

construction [13], which differ from authority for perform surveying work in field of real estate cadastre.

The purpose and the goal of this procedure is to define the content, form, the procedure of preparation, authorization, data security, audit review and control of the produced geodetic elaborates and projects from engineering geodesy. Procedures should be carried out in a unique way and in accordance with the formats and forms which are defined by the bylaws of the Law for real estate cadastre and the Law for construction.

The application of this procedure is in areas of settling disputes and other property rights of real estate owners in AREC, such as preparation of project documentation in accordance with Law for construction.

3.1.2 Procedure - Valorization of real estate market value

The valorization of real estate market value is performing by Law for valorization, which prescribes the methodology, rules and standards for valorization. It includes recognized international standards for valorization, and other laws that regulate value valorization of different forms of ownership and regulations based on them.

The valorization of real estate market value may be required in cases for transfer of ownership rights, financing and raising mortgage and lien credits, procedures for expropriation, property taxes and other cases provided by law. The purpose and the goal of this procedure is to define the content, form, the procedure of preparation, authorization, data security, audit review and control of the produced reports for valorization of real estate market value.

3.1.3 Procedure - Court expertise for real estate

Expertise connected with real estate property represent model of prepared expertise report in which is included application of scientific methods and knowledge, technical achievements and experiences gained in the field of geodetic science and real estate cadaster [12]. Expertise report and opinion is required in cases and procedures to resolve and proving ownership rights on a specific property in proceedings in front of judicial institutions.

4. IMS QUALITY POLITICS IN PRIVATE GEODETIC COMPANIES

IMS politics should be incorporated into overall business strategy of the geodetic company, in order for permanent improving the work quality of the company, protection and improvement of the environment, maintenance and continuous improvement of health protection and safety at work [3].

The basic standpoints of IMS in geodetic companies should be recognizable like feedback of quality and reliability of services and also satisfaction and confidence among clients. Providing continued improvement of the work quality processes should be the responsibility of all employees, but mostly of company management.

By incorporating preventive measures to the working processes and regarding to preserving the environment, geodetic companies affect the preservation of natural resources and reduce waste and detrimental emissions of harmful substances.

In this context it is necessary to implement training and motivation of employees, to perform their duties in connection with environmental policy, compliance and enforcement of regulations and standards for environmental protection, with a sole purpose company to be environmentally responsible organization.

The essence of the system for managing of occupational health and safety should be an identification of hazards, risk assessment and preventive measures for safety and health of employees. The working processes should completely be in accordance with the legal regulations which regulate the protection and safety, as well as the internal standards of the organization.

The IMS quality politics should be sustainable based on its applicability and continuous improvement to market oriented working system. The primary goal is completely meeting the requirements, needs and expectations of the clients or users of the services offered by the private geodetic companies.

With a view to continuous improvement as a basic condition for survival and development of companies and the continuous improvement of IMS, there is a need for planning processes and measures to promote and implement quality objectives.

5. CONCLUSIONS

Integrated management quality systems shouldn't be ignored and neglected, and they should necessarily be applied to companies that pretend to be serious factors within the activity that have on the domestic and international market.

More emphasized globalization of the market, continuous changes that are happening in organizations and in their environments, from technological, and also from economical aspect are increasing the need managers to think how to survive their companies and how to be competitive in a global market, where competition is without mercy.

The implementation of systems for quality management, besides other economic factors (business conditions, legislation etc.) that affect the operation of companies, is a stage of development when the overall operations and organization in the company rises to a higher level.

The benefits of properly implementation and application of the standards ISO 9001 (QMS), ISO 14001 (EMS) and OHSAS 18001, in an integrated IMS, are predominate over the disadvantages that are minor, if the implementation of standards is in right moment.

The financial results, increasing of profits, reduction of losses, costs, and increased productivity and competitiveness, continuity in the quality of products and services, penetration of new markets, are only part of the series benefits from the implementation of systems for quality management.

Despite increasingly need for recognition, increased competition and other economic factors that are affecting the working process of geodetic companies in the Republic of Macedonia, is still a small number of geodetic companies in ten years period of private geodetic practice that continually, (beside their professional and technical advancement and development) are following and are improving its development with economic component from aspect of marketing, management and implementation of management systems in their organizations.

The implementation of management systems, among other economic factors in the future will certainly represent an imperative for success-ful working and sustainable development of each geodetic company. Furthermore, these systems are a factor which is affecting the improvement and enhancement of working quality, protection and improvement of the environment, maintaining and continuously improving the protection of occupational health and safety.

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