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EDITORIAL - Preface to Volume 5 Issue 2 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 biannually 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, geotechnics, 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, it is my pleasure to introduce the Second Issue of VOLUME 5.

The year that is approaching its end was very unpredictable; it had surprised us with several natural disasters and opened several topics for reflection. Macedonia faced terrible floods in August and the very next month one frightening earthquake and several aftershocks have jolted Skopje and its surrounding. Therefore, this issue includes 5 articles dedicated to the natural disasters and continuous human struggle with them.

The first paper presents research and conclusions of an interdisciplinary review of possible country-level alternatives for adopting flood risk management in Macedonia, proposing institutional model based on purely engineering standards and ad-hoc interventions to flood events with an integrated risk-based flood management. The second paper describes in detail the investigation of the hydrological model for the exceptional flood event occurred on November 4th 1966 in the Arno River basin in Tuscany in Italy, in the occasion of its 50th anniversary.

The third and fourth papers consider seismic risk assessment of structures. The third paper is investigating vulnerability curves of bridges, while the fourth one optimizes the response of the frame structures due to earthquakes.

The final paper points out the importance of designing buildings capable to withstand different fire scenarios that can cause significant damages to their structural elements.

We cannot stop natural disasters but we can arm and prepare ourselves with knowledge. Keep your spirits and determination unshaken. Armed with courage, faith and efforts, you shall conquer everything you desire.

I wish you a very Happy New Year 2017 and Merry Christmas!

Sincerely Yours,
Assoc. Prof. Dr. Todorka Samardzioska
December, 2016

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INTRODUCING FLOOD RISK MANAGEMENT

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The evidence worldwide is that people will not, and in certain circumstances cannot, abandon flood-prone areas − whether they are in the sparsely populated floodplains or in the mountains. Therefore, there is a need to find ways of making life sustainable in the floodplains − even if there is considerable risk to life and property. The best approach is the integrated management of floods. This paper presents the findings of an interdisciplinary review of possible country-level alternatives for adopting flood risk management in the Republic of Macedonia, by harmonizing the national water management, and other related systems with the requirements of the EU Floods Directive. The proposed, most feasible, tailor-made institutional model aims to replace the traditional flood management approaches based on purely engineering/design-based standards and ad-hoc interventions to flood events with an integrated risk-based flood management. Given the similarity of existing flood management systems in the countries of the wider region, these findings can be used for initiating similar improvements in line with the contemporary flood risk management principles.

Keywords: Flood Risk Management, EU Floods Directive, Institutional Models.

1 INTRODUCTION

Societies, communities and households seek to make the best use of the natural resources and assets available to them in order to improve their quality of life. However, they are all subject to a variety of natural and man-made disturbances such as floods, droughts and other natural hazards, economic recessions and civil strife. These disturbances adversely affect personal assets and the community, such as job availability, natural resources and social networks.

The evidence worldwide is that people will not, and in certain circumstances cannot, abandon flood-prone areas − whether they are in the sparsely populated floodplains or in the mountains. Therefore, there is a need to find ways of making life sustainable in the floodplains − even if there is considerable risk to life and property. The best approach is the integrated management of floods.

The EU Floods Directive (2007/60/EC) is legislation in the European Parliament on the assessment and management of flood risks. Its aim is to reduce and manage the risks that floods pose to human health, the environment, cultural heritage and economic activity. The Directive requires Member States to first carry out a preliminary assessment to identify the river basins and associated coastal areas at risk of flooding. For such zones they would then need to draw up flood risk maps and establish flood risk management plans focused on prevention, protection and preparedness.

This Directive's implementation needs to be harmonized with the Water Framework Directive (2000/60/EC), by coordinating flood risk and river basin management plans, and through coordination of the respective public participation procedures.

The harmonization of the existing national water management system in the Republic of Macedonia with the requirements of the EU Floods Directive provides the opportunity for replacing the commonly applied ad-hoc responses to flood events and the traditional approaches based on purely engineering/design-based standards with an integrated risk-based flood management.

This paper presents the findings and recommendations of the UNDP-backed interdisciplinary review of possible country-level flood risk management models aligned with the approach of the EU Floods Directive.

2 OVERVIEW OF HYDRO-METEOROLOGICAL HAZARDS IN THE REPUBLIC OF MACEDONIA

Macedonia is highly exposed to flooding, drought, extreme temperatures, forest fires and earthquakes. Seismicity is high and, according to a recent World Bank study, between 1990 and 2008 only three countries in Europe and the CIS (Commonwealth of Independent States) experienced more climate related natural disasters (The World Bank. 2009. Adapting to Climate Change in Eastern and Central Europe).

Floods are the chief hydro-meteorological hazard. Flood frequency and intensity are rising. River floods in the major river basins are caused by long periods of rainfall and the intensive snow melting. Flash floods caused by short intensive rainfall (most frequently by summer storms) can occur in smaller river basins. River floods occur in the basins of the Vardar, Crna Reka, Treska, Strumica, Pchinja, Lepenec and Bregalnica rivers, Fig. 1. The Vardar River basin, which is the largest in the country, accounts for 80% of the water resources and experiences the highest level of exposure to flooding. The floods of 1962 and 1979 resulted in economic losses of approximately 7% of GDP (for each year), while economic losses resulting from the floods in 1994 amounted to 3.4% of GDP.

Flooding in 2004 affected 26 municipalities (mainly in the area of the upper Vardar, but also in the central, southern and south eastern parts of the country), with estimated damage of approximately USD 21 million, primarily in the agriculture sector [9].

Meteorological and hydrological drought is also common in the Republic of Macedonia. The most highly exposed agricultural zones are the Povardarie region (especially in the area of the confluence of the Crna and Bregalnica rivers with the River Vardar); the south eastern part of the country, the southern Vardar valley, the Skopje-Kumanovo valley and Ovche Pole. A prolonged drought in 1993 wiped out much of the crop yield, damaged livestock and resulted in economic losses that comprised 7.6% of the total national income.

Extreme temperature and heat or cold waves not only have a direct impact on the health of humans and animals (e.g., diseases affecting livestock) but also secondary impacts that can create the conditions for wildfires or lead to soil aridity, which makes certain areas more prone to mudflows. The year 1994 stands out in the period 1971-2000 as the hottest year on record in the country, with temperatures 2oC above the multi-annual average. High temperature anomalies were also registered in 1999, 2002 and 2003. Extreme air temperatures in July 2007 exceeded all other previously registered temperatures, with 45.7ºC in Demir Kapija, 45.3ºC in Gevgelija and 43.4ºC in Skopje.

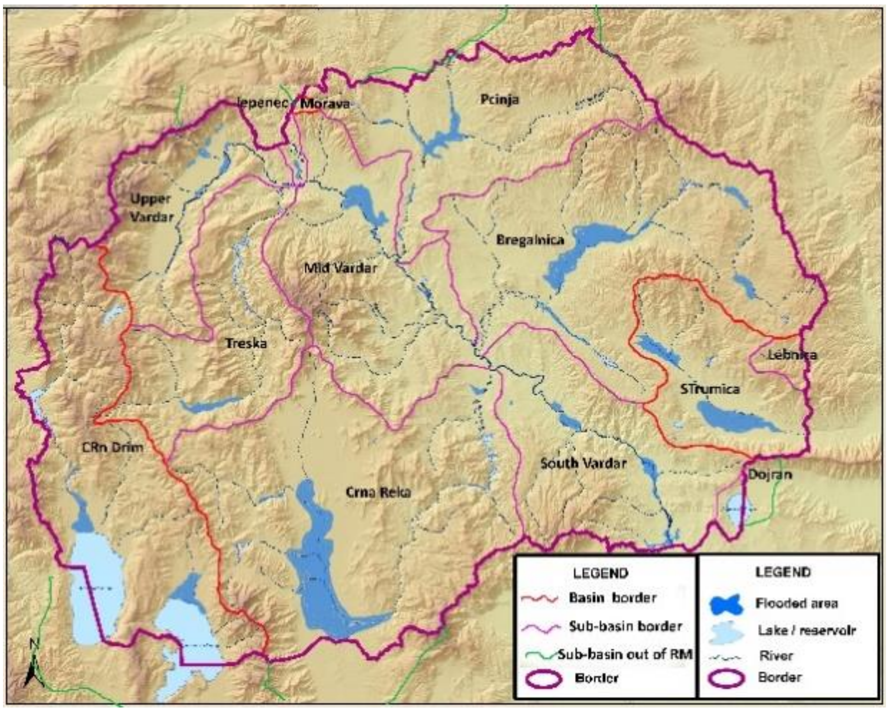

Figure 1. Most frequently flooded areas in the Republic of Macedonia

3 HISTORY OF FLOODS

Intensive rainfall and increase of groundwater levels, combined with poorly maintained flood control infrastructure, result in frequent flooding of flat, mainly former wetland areas, such as Polog, Pelagonija, Skopsko Pole, Strumichko Pole.

Traditionally, the main activities in urban rivers comprised conventional regulations for improved flood control. In the majority of cities, rivers have been regulated mainly by considering hydrological and hydraulic aspects, with very little or no attention to ecological functions and aesthetic aspects.

A typical example of such “functional” regulation, often referred to as a good practice from the past, has been the regulation of the Vardar River in the country’s capital Skopje (Fig. 2, left). However, the flood control functionality of the regulated riverbed has been seriously jeopardized with the implementation of the controversial “Skopje 2014” project. This massive urban infrastructure development project in the centre of Skopje comprised various new structures in the major riverbed, construction of new low placed footbridges with insufficient discharge capacity, construction of fountains and plant boxes in the basic riverbed, construction of fixed concrete/steel boats, also in the basic riverbed, as well as additional structures in the river’s floodplain (Fig. 2, right). Such development happened in time when majority of EU water-related legislation has already been formally adopted – although its practical implementation is challenged by multiple barriers – institutional, financial and human capacities.

The adoption and operationalization of EU Floods Directive – the only piece of EU water-related legislation that hasn’t been incorporated in the national systems yet, would be instrumental in preventing future similar developments that increase flooding risks.

The genesis of floods occurring in Skopje is very specific mainly due to coincidence of flood wave peaks in Upper Vardar, Treska and Lepenec rivers. Because of this the city of Skopje was very often flooded and the most catastrophic appeared in 1778, 1858, 1876, 1895, 1903, 1916, 1935, 1937, 1962, 1979. The floods of 1962 and 1979 resulted in economic losses of approximately 7.2-7.4% of GDP (for each year), while the floods in 1994 caused losses of 3.4% of GDP. Flooding in 2004 affected 26 municipalities.

UNDP Report (1993-2007) registers seven floods affecting 111,400 people and causing an estimated damage of around 353,600 US$. The occurrences of extreme hydrological events (floods and droughts) have increased in frequency and intensity over the past decades as a response to changing climate conditions. For example, during the last three decades regional floods caused by the biggest rivers in Macedonia, Vardar, Crna Reka, Strumica, Treska, Pchinja, Lepenec, and Bregalnica, caused an estimated total damage worth 193.8 million US$.

Figure 2. Actions against flood risk management concepts-regulated Vardar River in Skopje in 2010 (left) and in 2014 (right)

Only in June 2004, intensive rainfall, caused floods and torrents affecting 26 municipalities (mainly in the area of upper Vardar, but also in the central, southern and south eastern part of the country) with estimated damage of 15 million EUR. Most of the damage from floods was caused in rural areas by flooding households and arable land. Concerning the impact of climate change on Macedonia's water resources and extreme hydrological phenomena, the risks from intensive torrents and prolonged droughts are expected to increase. The damage caused by floods directly affects the already fragile agriculture and local rural economies. Economic losses experienced during the flash floods in 2004 show that 91.3% of the total damage is attributed to the agricultural production mainly in the south-eastern part of the country. The biggest losses have been experienced in the rural areas where households and cultivated areas have been flooded.

Severe flooding hit much of the country in January and February 2015, causing widespread damage and economic losses. Heavy rainfall caused rivers to overflow in many locations, and 44 out of 80 municipalities experienced floods. The most affected regions were the basins of the Crna Reka, Bregalnica and Strumica rivers, which cover 45% of the territory of the country. Roughly 170,000 people were affected in all. The floods caused major damages to roads and bridges, interrupting transport. Much agricultural land was also flooded, causing extensive losses to farming families. Drainage and irrigation systems were damaged, and private houses, private-sector industrial facilities, schools and public facilities in some villages were flooded.

The initial impact assessment estimated the total cost of the spring 2015 floods at 35,691,672 EUR. Of this total, 62 percent were classified as damages and 38 percent as losses.

A subsequent flood-related disaster hit the country on 3 August 2015, when flash floods and mudslides struck the northwest Polog Region, killing six people and causing damage to municipal infrastructure and houses in the city of Tetovo and villages in the surrounding mountainous areas. The total damages from these floods are estimated at 21.5 million EUR.

Most recently, the night between 6 and 7 August 2016 heavy torrential rain affected Capital's, causing tragic loss of 23 lives and dozens injured or missing, Fig. 3 and Fig. 4. The estimated cost is over 30 million EUR, resulting from the severely damaged infrastructure and affected agricultural land. The Hydrometeorological Service measured 93 liters per square meters of rain fell in just a few hours, which is considered an event with a return period of 1,000 years.

The tragic consequences of this event, the magnitude of the damage and the response challenges in its aftermath, emphasize the need of urgent reforms in the flood management system in line with the contemporary risk-based management approaches.

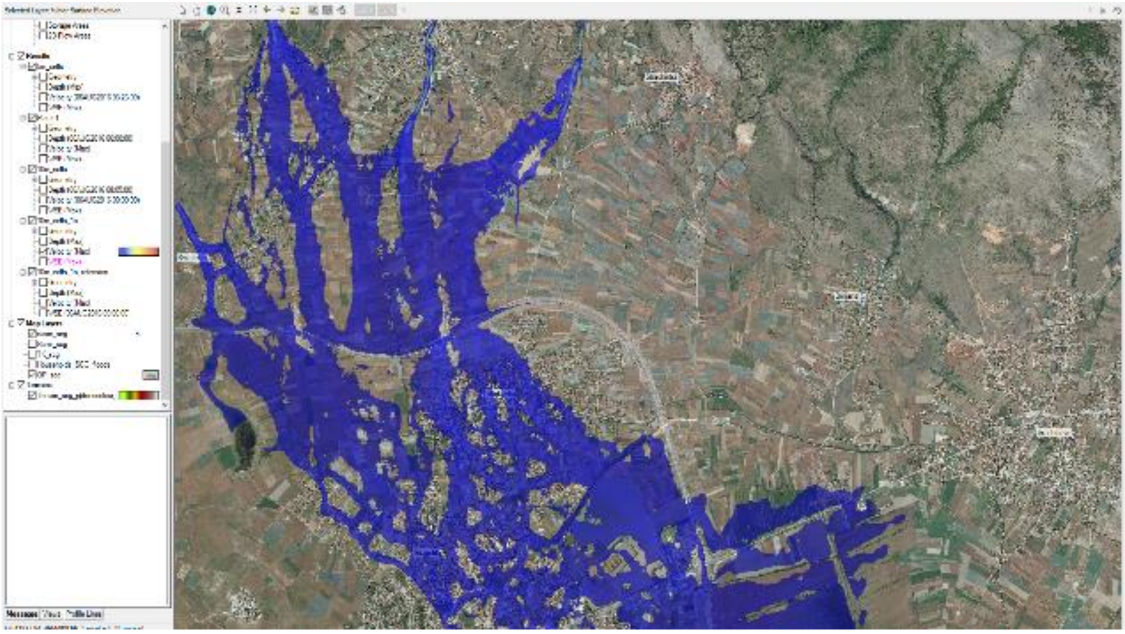

Figure 3. Skopje flood on 6 August 2016: 2D hydraulic simulation of the flooded area

Figure 4. Skopje flood on 6 August 2016: the effects of flooding

The apparent increase in the frequency and intensity of flood events indicate that country's vulnerability is increasing. Floods participate with 44% of all disaster events in the Republic of Macedonia [9]. Registered flood events in the last 60 years are presented in Fig. 5.

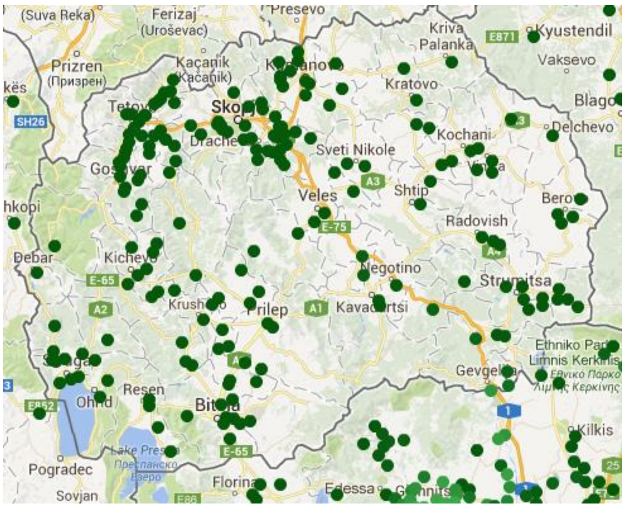

Figure 5. Registered flood events in the last 60 years

4 FLOOD MANAGEMENT CYCLE AND STRATEGIES

Floods are natural phenomena which cannot be avoided. However, it is possible to reduce their negative consequences to certain extent. Traditional "Flood protection" aims at preventing flood hazards up to a certain level by providing a defined protection (or safety) standard (for example: 100 years return period). Such protection levels are mostly established by means of flood defense structures such as dikes, dams, retention areas, etc. In recent years, traditional concept of "Flood protection" is being transformed to new concept of "Flood Risk Management" [7].

This section presents the basic flood management concepts and strategies upon which the functional analysis of the proposed institutional models has been carried out.

The concept of Flood Risk Management (FRM) tries to adjust flood protection to the risk situation by concentrating protection efforts to areas with a high expected damage, in order to spend public funds in an economically efficient way [6].

Therefore a risk-based approach is to achieve the best management results possible by using the budget and resources available. Flood risk is defined as product of the likelihood or chance of flooding, and the adverse consequences or impacts of flooding (where negative consequences cover social, economic, environmental and other impacts), Fig. 6.

Elements of flood management are closely related to flood management strategies, as for example defined in Table 1.

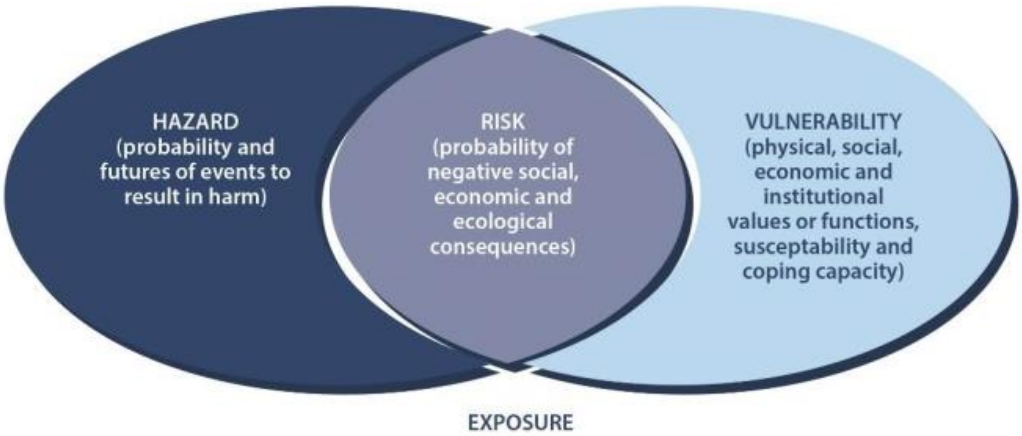

Figure 6. Risk as inter-relation of hazard and vulnerability (*Schanze*, 2009)

Table 1. Flood management strategies

Strategy	Options
Reducing Flooding	dams and reservoirs
	dikes, levees and flood embankments
	high flow diversions
	catchment management
	channel improvements
Reducing Susceptibility to Damage	floodplain regulation
	development and redevelopment policies
	design and location of facilities
	housing and building codes
	flood proofing
	flood forecasting and warning
Mitigating the Impacts of Flooding	information and education
	disaster preparedness
	post-flood recovery
	flood insurance
Preserving Natural Resources of Flood Plains	floodplain zoning and regulation

Source: Integrateted Flood Management Concept Paper, WMO, 2009.

The so called "cascade of measures", Fig. 7, can be used as an elementary guidance for planning the FRM measures based on the source-pathway-receptor principle which include: (i) reduction of the flood source (reducing of runoff) to prevent high discharges and high flood risks downstream as the most favourable measure, (ii) reduction of the hydraulic load on flood control structures by reducing and transforming flood wave discharges and water elevations, (iii) conventional flood control measures, (iv) zoning measures to help reducing the potential impact, (v) impact reduction measures, such as flood proofing of houses, early warning and evacuation, and (vi) residual risk reduction measures, where other measures are not sufficient.

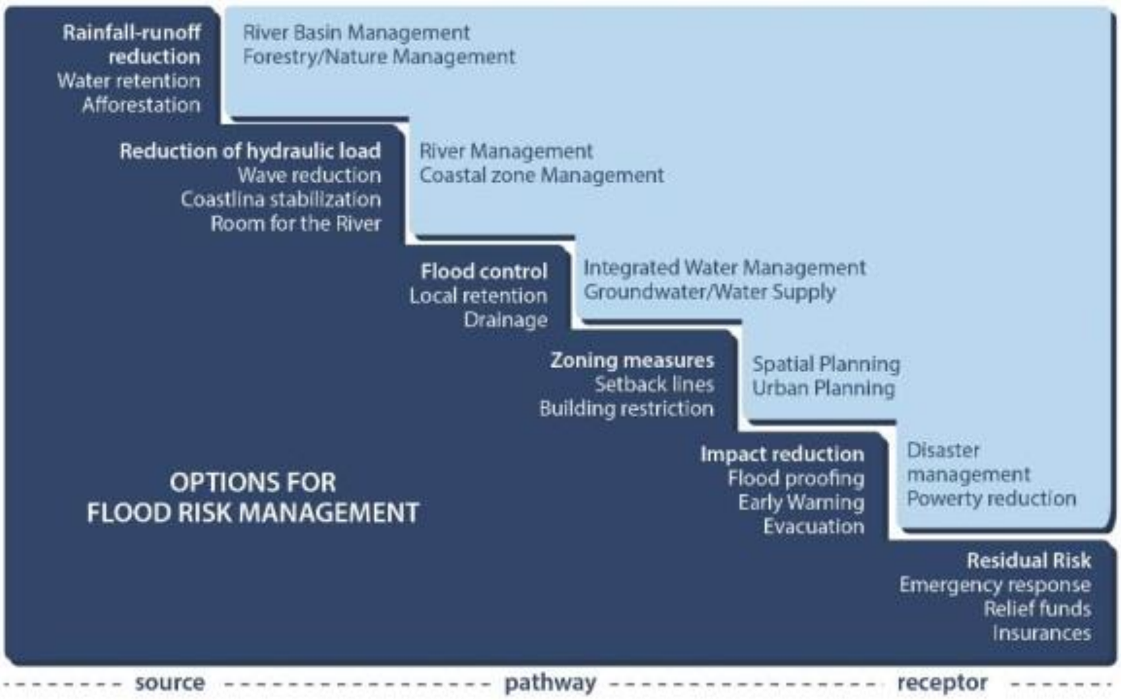

Figure 7. Cascade of measures

5 ANALYSIS OF POSSIBLE ALTERNATIVES

Flood risk management comprises significant number of different activities interacting with numerous policies and activities. For the purpose of the analyses presented, the flood risk management has been approached to as a public service with significant application of solidarity principle.

The EU Floods Directive introduces systematic approach primary to flood risk management planning aspects and integrated water management/governance, while operative flood defence is not placed in focus. However, it provides significant freedom for shaping flood risk management to suite national needs and thus, in a broader sense, it includes operative flood protection in flood risk management.

The institutional arrangements are not unified and differ significantly among EU member states. Having in mind that the current institutional setting in Macedonia is not completely defined and the responsibilities are distributed across different bodies, there is an opportunity to develop a tailor-made flood risk management system. The proposals take into consideration the existing country systems with certain competences over the management of floods (e.g., Ministry of Environment and Physical Planning which is also the EU WFD body, Hydro-meteorological Services – HMS, Crisis Management Center – CMC and Protection and Rescue Directorate – PRD, both of them with responsibilities over the operative flood defence and multi-hazard planning approach). Considering country's limited resources, area and population (area of 26,000 km2, population of 2,100,000) the approach to institutional improvement of flood risk management is suggested to follow one of two possible directions: (i) introduction of strong coordination mechanisms or (ii) centralisation of (at least some) management functions. Although, possible approaches, with sub-options are numerous, four basic alternatives have been considered:

1. Body dedicated to flood risk management (FRM)
2. Flood risk management (FRM) integrated with Water Framework Directive (WFD) body
3. Flood Directive (FD) activities integrated with WFD body
4. Flood Directive (FD) activities implanted in existing institutions

Alternative 1: Body dedicated to flood risk management (FRM)

This body consists of 6 units, corresponding to the key necessary functions (planning, development and implementation, operative flood defense, public and stakeholder relations, information support), as well as management and logistics. It entails all main functions of flood risk management. However, integrated water management is not implemented within this body.

Figure 8. Body dedicated to flood risk management

Alternative 2: Flood risk management (FRM) integrated with WFD

In this option, in comparison to first one, one further step is made towards full integration of flood risk management in river basin management according to WFD. Considering the fact that certain activities of FRM and RBM overlap, they can be aggregated to achieve mutual benefits.

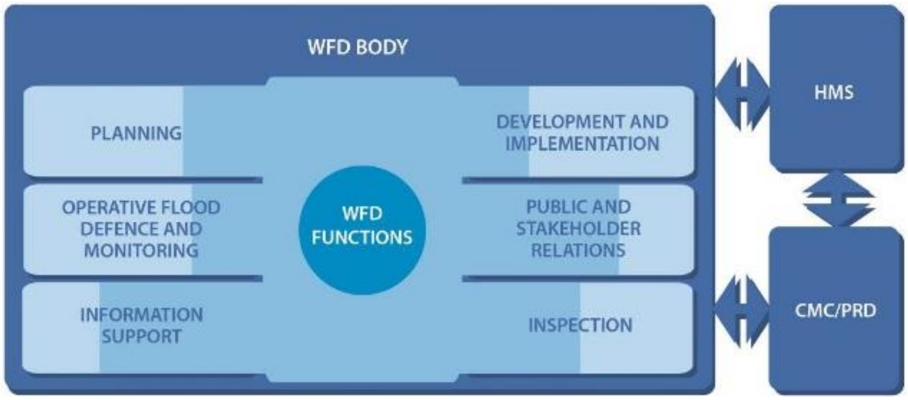

Figure 9. Flood risk management integrated with WFD body

Alternative 3: Flood Directive (FD) integrated with WFD

This option is same as option two, except that the operative flood defense is placed outside this body. In such case development of operative flood defense and supporting activities would be entrusted to the Hydro-meteorological Service (HMS), the Crisis Management Centre and the Protection and Rescue Directorate (CMC/PRD). Main benefit for FD/WFD body would be simpler organisation, smaller scope of work (e.g., no need for operative flood forecasting activities, and other activities supporting operative flood defense, such as maintaining of materials for flood fighting, ensuring additional workforce and equipment). Certain functions such as monitoring of structures and watercourses should be preserved for maintenance purposes.

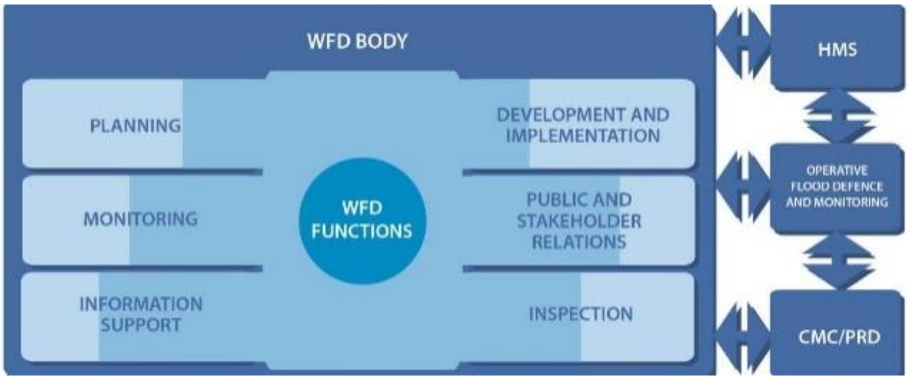

Figure 10. Flood Directive activities with WFD body

Alternative 4: Flood Directive (FD) implanted in existing institutions

This option relies on preserving existing instruments and enhancing inter-institutional cooperation. However, it should be noted that in present state, significant number of institutions on different levels are involved, urging for extensive cooperation and coordination.

At the same time certain functions, such as planning in line with EU Floods Directive are missing and still need to be implemented. Therefore, in order to achieve functional flood risk management significant institutional rearrangements should be done, and some kind of inter-institutional body for collaboration, international cooperation, public participation, reporting and setting joint standards needs to be introduced. The main goal of this option (non-disturbance of existing institutional arrangements) cannot be fully achieved if an effective flood risk management is to be introduced.

Table 2. Numerical assessment of the proposed alternatives

CRITERIA GROUP	CRITERIA	Weights	VALUE from 1- poor to 5-excellent			
			Body dedicated to flood risk management	Flood risk management integrated with WFD body	Flood Directive activities integrated with WFD body	Flood Directive activities implanted in existing institutions
FD RELATED	river basin approach	2.00	5	5	5	1
	integrated water management		2	5	3	1
	integration of FRM in RBMP		3	5	4	1
	international cooperation		3	5	3	1
	competent authority		3	5	4	1
FRM RELATED	unified approach to risk management	1.50	5	5	1	1
	conflicts of FD and operative flood defense		5	5	2	3
	solidarity principle		5	5	3	1
	efficiency of financing		3	5	4	1
	non-structural measures planning		3	5	4	1
ORGANIZATIONAL	clear responsibilities and relationships	1.00	5	4	3	1
	number of additional personnel		3	4	3	5
	concentration of expertise and knowledge		4	5	3	1
	institutional rearrangement		3	1	2	3
	logistics benefits		2	5	4	1
RESULT	average		3.60	4.60	3.20	1.53
	weighted average		3.58	4.73	3.29	1.40

6 CONCLUSION

From the flood risk management point of view, it is obvious that most suitable option is Alternative 2: FRM integrated with WFD. Only this option has the potential to fulfill all major *requirements* set by EU water management policies. Considering the complexity and sophistication of such body it has to be very carefully managed, improved and adapted to a particularly dynamic environment. Also, activities of this body are under competences of several Ministries which need to be represented in a management board/council.

The institutional model can be further adjusted in light of institutional development priorities, capacity strengthening needs and opportunities, as well as financial instruments in place. However, in any scenario, there are a number of preliminary activities whose implementation would be beneficial regardless on the selected option. These include, for example, development of a GIS-based spatial database on floods and flood management practices, development of flood hazard and flood risk maps, as well as flood risk management plans for the main river basins and priority areas in the country, and their use in spatial and urban planning. The introduction and development of such flood management approach is a continuous process which requires specific interdisciplinary expertise, coordination and communication instruments and major awareness raising. Therefore, strong focus needs to be placed on education, raising awareness at all levels in society and developing of efficient coordination mechanisms within institutions and communication with the population, especially in the areas at the highest risk of flooding.

Although complex and potentially costly, such an institutional reform can be easily justified as an absolute necessity if optimal flood risk mitigation and prevention of significant future losses is to be achieved in the country.

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THE FLOOD HISTORY OF ARNO RIVER: THE EXCEPTIONALITY OF THE EVENT ON NOVEMBER 4TH 1966

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The paper aims to describe the exceptional flood event occurred on November 4th 1966 in the Arno River basin in Tuscany (Italy), in the occasion of its 50th anniversary. The flood history of Arno River, with a particular attention to the last two millennia, is also reported. Particularly, a classification of the flood events occurred since the XII century, based on the procured damages, is described. Eight flood events have been defined as exceptional. The recent exceptional and more famous flood occurred on November 1966. The flood event is investigated in detail here, with reference to the meteorological causes that determined the exceptionality of the event. The results of a hydrological model are discussed in the conclusions to show the flood hydrograph and its peak flow at Firenze Uffizi urban cross section. The estimation of the economic damage suffered by the various districts of Arno river basin after the flood is also considered.

Keywords: flood, flood risk, hydrological model, Arno River, damage.

1 AIM OF THE PAPER

In terms of flood risk issues, the Arno River basin represents a particular case, well known by the hydraulic scientific and technical community. Most of its territory is prone to frequent flood hazards, with high levels of risk due to the vulnerability of unique cultural and artistic heritages.

In this context, the aim of this paper is to describe the floods of Arno River, occurred over the years in the last two millennia, focusing the attention on the exceptional recent event of November 4th 1966. The meteorological causes that determined the exceptionality of that event are analyzed and described, with a particular attention to rainfall intensity. The rainfalls measured in some stations within the event are compared with the rainfalls of the same durations ever recorded. For some of the stations, also the return period and the probability of occurrence are estimated in order to emphasize the exceptionality of the event. Water levels recorded during the event in significant hydrometric stations and the relative peak discharges are compared with the previous corresponding maxima.

Moreover, to acquire indications on the behavior of the river and the water levels that were reached in the event, a hydrological model is developed in order to obtain the flood hydrograph at the urban cross section of Firenze Uffizi.

Finally, using a historical survey approach, the estimation of the economic damages caused by the 1966 flood event in the different districts of Arno River basin is considered.

2 THE ARNO RIVER BASIN

The Arno River is almost entirely situated within the Tuscany region in Central Italy. The river is 241 *km* long. It originates on Mount Falterona, at 1385 *m* a.s.l., in the Casentino area of the Apennines. It turns to west near the city of Arezzo, and passing through Firenze, Empoli and Pisa it flows into the Tyrrhenian Sea. The catchment area is about 8830 km^2, and it has a mean elevation of 353 *m* a.s.l.. The physiography of the catchment is strongly influenced by the morphology of the region, which is characterized by a series of intermountain basins, alternated with bedrock-controlled gorge-like reaches.

3 HISTORY OF THE ARNO RIVER FLOOD EVENTS

Throughout its history, the Arno River has frequently inundated Firenze, causing often catastrophic damages. Over the centuries, the relationship between the city and the river was undoubtedly tormented, because the latter allowed the birth of the Florentine population, but, at the same time, its fury has often led to numerous casualties.

The original historical data of Morozzi, published in 1762 [14], integrated with data by different sources, mainly of the Regional Administration, are summarized in Figure 1, and show the distribution of flood events that caused damages to the city of Firenze between the 12th and the 20th century. Morozzi recorded and classified each flood, between 1173 and 1761, into three magnitude levels on the basis of damage caused: in particular, floods are classified as medium, large and exceptional. Eight floods of those considered can be defined as exceptional, but it is worth noting that Firenze was also inundated 55 other times, among which 24 can be consider large, because of the consequences.

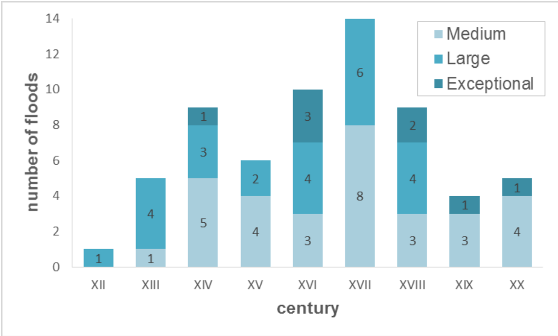


Figure 1. Distribution of the flood events of the Arno River in Firenze, between the 12th and the 20th century, ranked by intensity based on the caused damages

The first recorded flood of the Arno River occurred in 1177 [15]. However, the most remembered is the one of November 4th 1333: the inundation caused about 300 fatalities [17], Ponte Vecchio was completely destroyed and it was rebuilt in 1360 exactly as it is now.

Three exceptional floods occurred in the XVI century, in 1547, 1557 and 1589, once again causing hundreds of casualties and devastations in the Santa Croce area. Other two major events took place in the XVIII century, in 1740 and 1758, but after the period investigated by Morozzi, only two exceptional and catastrophic floods occurred in Firenze, one in 1844 and the other in 1966.

A color painted map of 1847 by Manetti, reproduced in Figure 2, shows the extension of the inundated areas within the Arno river drainage basin, during the event of the 1844. It clearly demonstrates the high magnitude of that event and the strong impact that it had on the social activities located in the flooded areas. The extension of the latter, in 1844, was comparable with that of 1966. Instead, the impact in terms of losses was much higher in 1966 because of the increased exposure and vulnerability of the elements at risk. The floods of 1557 and 1966 are two of the three most serious floods occurred after the one of 1333, which seems to be perhaps the worst as number of victims. The peak flows at Nave di Rosano, upstream the city of Firenze, reached in the three cases more than 4.000 m^3/s, while in 1844 it was about 3.000 m^3/s [16].

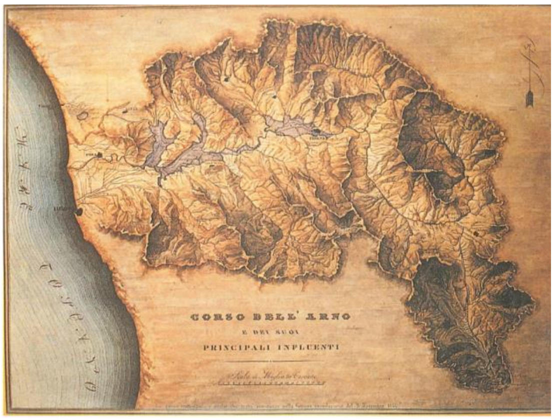


Figure 2. Extension of the areas inundated by the Arno River on November 4th 1844, from a color painted map of Manetti dated 1847

4 THE NOVEMER 4TH 1966 FLOOD

The flood occurred on November 4th 1966 is undoubtedly the most catastrophic ever recorded (Figure 3). It was produced by an exceptional meteorological event between the 3rd and the 5th November 1966, which affected the whole Italian peninsula.

The causes determining the exceptionality of the event were various and concomitant. Surely very important were the anomalous climatic condition of October, causing specific thermal and hygrometric characteristics of the air masses [10][11]. In the month of October 1966, in fact, and particularly in the third decade, the precipitations were abundant and continuous, in many regions where flood events occurred in November. Persistent rainfall over wide areas reduced the soil storage capacity and the aquifer receptiveness. The three days preceding November 3rd were extremely dry, but rainfall of November 3rd and 4th were exceptionally intense, incessant and extended. The heavy rainfall led to exceptional peak discharges in several rivers of the Italian Peninsula [10]. In Veneto region, values of the peak flow exceeded all previously recorded maximum value. In Tuscany, in some cross section of the Arno River the discharges were more than double of the maximum previously measured. In Emilia-Romagna as well as in various tributaries of the Po River, particularly in the Panaro River, the discharges exceeded the maximum data recorded until that time.

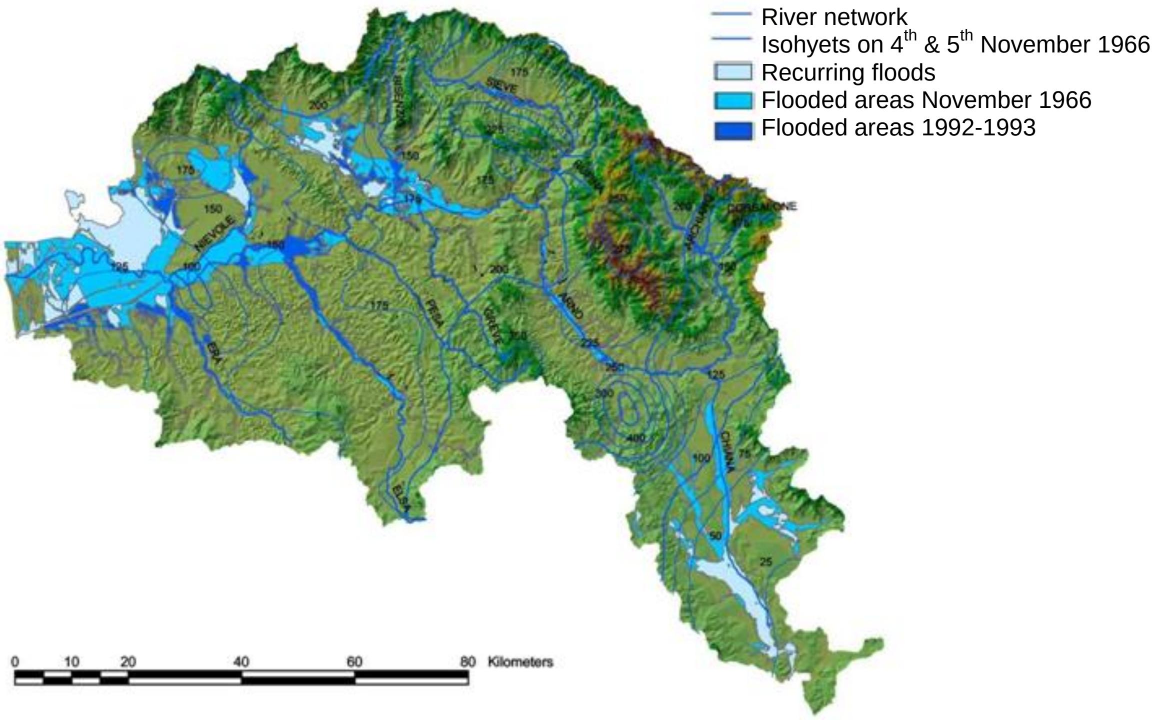


Figure 3 – Extension of the areas inundated by the Arno River flood in 1966 and 1992-93, with isohyets of the rainfall event of the 3-4 November 1966 [5]

The precipitation started in the early hours of November 3rd and became large and persistent since 11 a.m. to 12 a.m.; many peaks were recorded in central and southern Tuscany until 12 a.m. to 2 p.m. of November 4th. The event had an overall duration of about 26-28 hours.

The large size of the involved territory and the extension of rainfall are further key aspect that is worth considering. All major catchments of the Tuscany Region were affected by the intense rainfall and those most affected included the river basins of the Upper Arno valley, the catchments of the left bank tributaries, among which Greve, Pesa and Elsa, as well as the Ombrone River basin (Grosseto) and the catchments of Bruna and Pecora rivers.

The maximum daily precipitation in the Arno catchment was recorded at Badia Agnano rain gauge, in the Ambra River basin (a left bank tributary near the town of Arezzo), where a daily rainfall of 338,7 *mm* was recorded and 437,2 *mm* in 48 hours. In some Tuscany areas, the ratio of daily rainfall of the event to the maximum daily rainfall observed in the previous period was 200% and considering duration of two days it was even 250% (Figure 4).

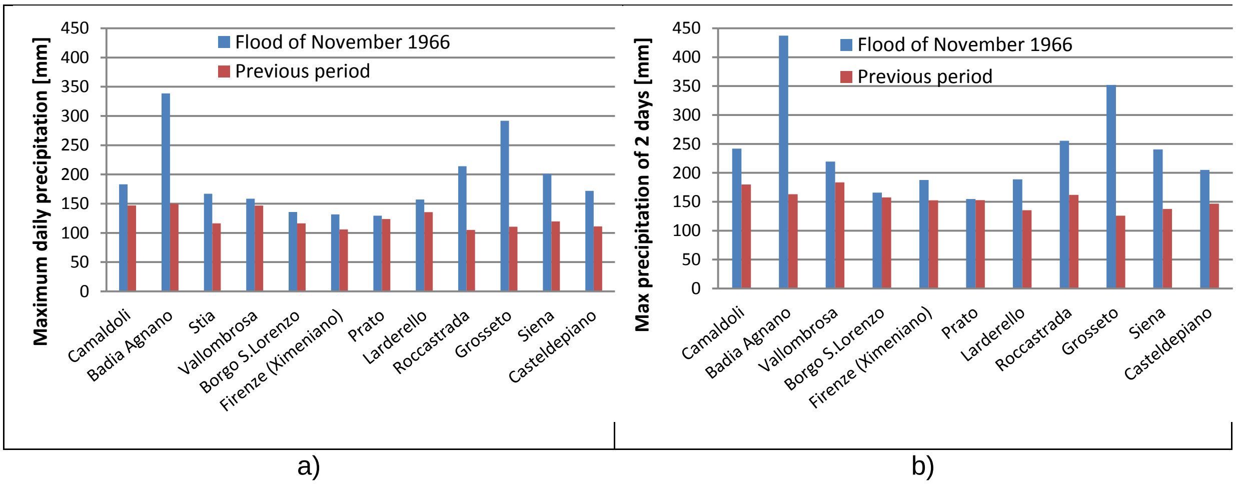


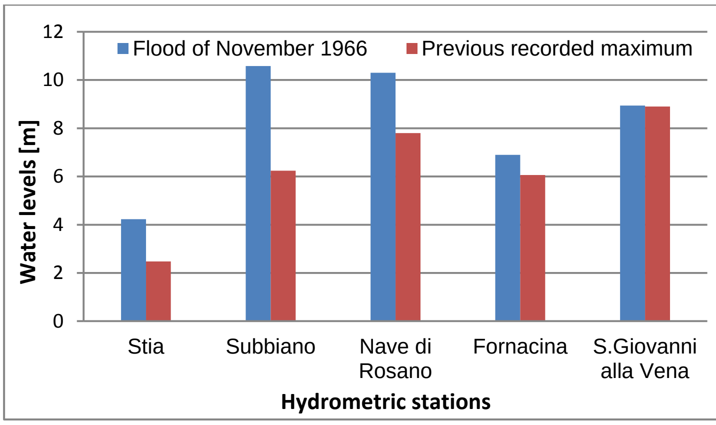
Figure 4. Comparison among rainfall data of some rain gauge stations of Tuscany region: a) daily precipitation relative to the flood event on November '66 and the maximum recorded in the previous period until 1965; b) precipitation of 2 days duration relative to the flood event of November '66 and the maximum recorded in the previous period until 1965 (Data source: Commissione De Marchi, 1970 [10])

In order to give a further idea of the exceptional nature of the event, the estimate of the return period and the probability of occurrence of the recorded rainfall were carried out using the methodology explained in the study of Caporali et al. [6] about the regional frequency analysis of extreme rainfalls in Tuscany. The higher precipitation was the one recorded at Badia Agnano (AR), which caused the extraordinary flood of the Ambra River: the return time of such event is estimated more than 1300 years, with a probability of occurrence of 0,07%. The return period of the precipitation recorded at Stia (AR) is about 500 years and the probability of occurrence is 0.17% (Table 1).

These heavy rainfalls led to exceptional peak discharges in many tributaries of the Arno River. Furthermore, as previously underlined, when the storms occurred, the river conditions were already critical and the hydrometers recorded water levels ever measured before. For example, the hydrometer of Stia recorded 4,23 *m*, almost double the maximum recorded in previous years (2,48 *m*), and at the hydrometer of Subbiano water level reached 10,58 *m*, exceeding of more than 4 *m* the maximum level previously reached (6,24 *m*) (Figure 5).

Table 1. Estimate of the return period and the probability of occurrence of the rainfall recorded on November 4th 1966 in some rain gauge stations in Tuscany Region, described by name and code (Figure 7) of the Hydrological Regional Service (www.sir.toscana.it), and the hydrographic sub-basin of location.

Code	Rain gauge station	Hydrographic sub-basin	Return period [Years]	Probability of occurrence
870	***Badia Agnano (AR)***	Ambra	>1300	0,07%
610	***Camaldoli (AR)***	Casentino	175	0,57%
580	***Stia (AR)***	Casentino	500	0,17%
640	***Salutio (AR)***	Casentino	50	2%
1000	***Borgo San Lorenzo (FI)***	Sieve	90	1%
1030	***Dicomano (FI)***	Sieve	120	0,82%
1340	***Montespertoli (FI)***	Elsa	90	1,13%



Hydrometric Station	catchment area [km^2]
Stia	62
Subbiano	738
Nave di Rosano	4083
S. Giovanni alla Vena	8186
Sieve at Fornacina	831

Figure 5. Comparison between the values of the water levels recorded during the flood event of November ’66 and the maximum level ever reached and recorded at significant hydrometric stations on Arno River (and Sieve River, right bank tributary). In the table, for each hydrometric station the catchment area is also reported (Data source: Commissione De Marchi, 1970 [10])

In the Arno catchment, all the different tributaries contributed to reaching of the flood peak discharges. The highest peak discharges occurred in the Sieve River, the major tributary of Arno River, with a catchment area of 846 km^2, on the right bank, as well as in the Elsa and Era rivers, left bank tributaries. In these latter, during the flood, numerous riverbank breakages occurred causing different floods. However, the flood wave was exceptional from the beginning [12]. For example, at Stia, despite the catchment area is only about 62 km^2, the peak discharge was 312 m^3/s, i.e. equal to 236% of the maximum ever recorded of 132 m^3/s. At the cross sections of Subbiano and Nave di Rosano the reconstructed peak discharges were about 2250 and 3540 m^3/s respectively, equal to 258% and 171% of the maximum ever recorded (Figure 5).

The results of a statistical analysis carried out by Paris and Solari [16] show that the return period of the event of November 4th 1966, when the peak flow at the Nave di Rosano cross section reached 4000 m^3/s, is about 230 years. While, downstream of Firenze, due to the floods in the upper part of the river, the water levels increased more slightly: at the last hydrometric station of San Giovanni alla Vena, the peak discharge was 2290 m^3/s, very close to the maximum previously recorded in 1949 at the gauging station (2270 m^3/s) (Figure 6).

Even today, for most of the hydrometric stations with a dataset prior to 1966, the historical maximum values are those reached in the event of November 4th.

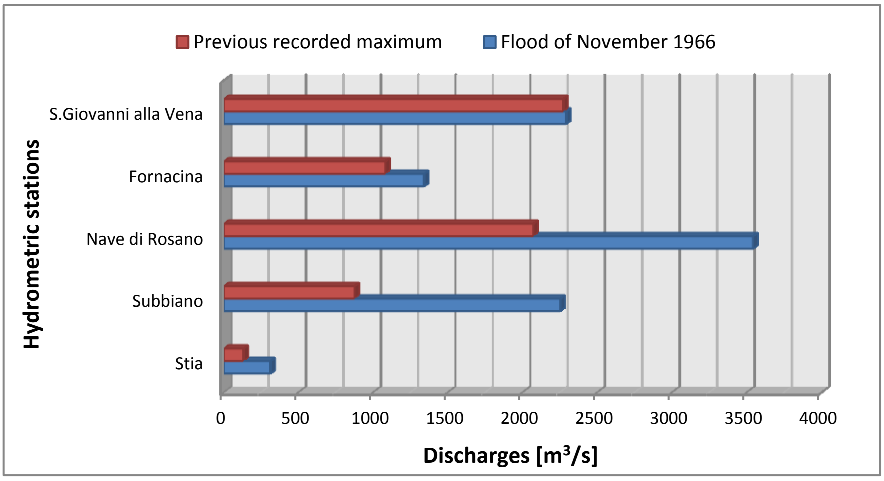


Figure 6. Comparison between the values of the peak discharges reached during the flood event of November '66 and the maximum discharges registered in the previous period at significant hydrometric stations on Arno River, for which also the rating curve was available (Data source: Commissione de Marchi, 1970 [10]).

5 HYDROLOGICAL MODEL

A hydrological model was developed using the software HEC-HMS to obtain the flood hydrograph at the river cross section of Firenze Uffizi (Figure), considered the closing cross section of the Arno river basin in the city of Firenze.

In the modeling was considered also the presence of the reservoirs of Levane and La Penna, with their outflow hydrographs during the event and the volume of flood pain of Laterina, as reported by the report of Cocchi et al. [9]. Because of the destruction of some hydrometers during the flood event, the measures in the hydrometric cross sections are often not available and data for the comparison of the measures with the results of the models are indirectly estimated from traces observed and recorded by the National Hydrographic Service at the different sections in the days after the event.

5.1 MODEL RESULTS

The modeling was performed considering a time span of 48 hours, from 12:00 p.m. of November 3rd to 12:00 p.m. of November 5th. The model was calibrated on the flood hydrograph of the event, reconstructed at the hydrometric station of Nave di Rosano [2][10] (Figure 7).

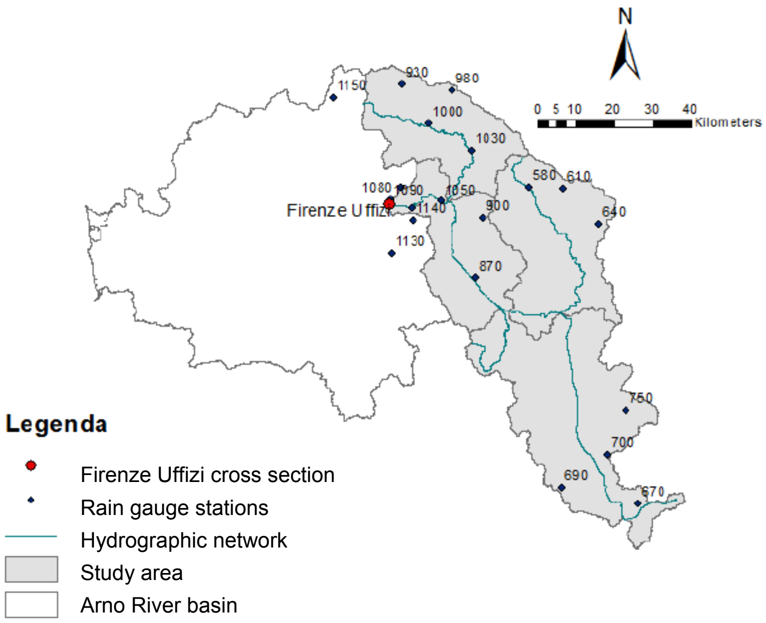


Figure 7. Hydrographic basin of the Arno River and delimitation of the study area, with location of the rain gauges and of Firenze Uffizi urban river cross section

The efficiency of the calibration is evaluated using the index of Nash and Sutcliffe. The obtained value equal to 0,983, shows a good accuracy of the model. Once checked the accuracy of the model, the flood hydrograph at the urban cross section of Firenze Uffizi, where the Arno River basin has an area of about 4230 km^2 is modelled and the estimated peak flow is about 4500 m^3/s (Figure 8).

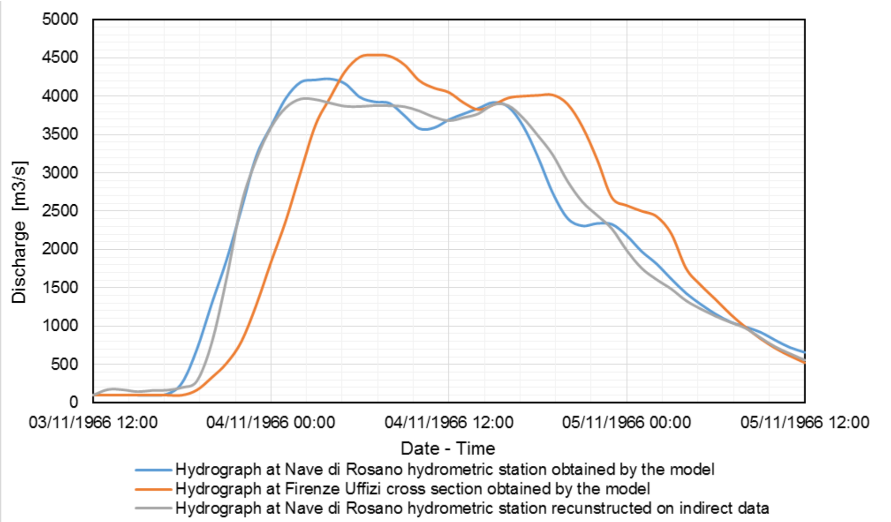


Figure 8. Comparison between the flood hydrograph at the hydrometric station of Nave di Rosano, obtained as result of the HEC-HMS model, and the hydrograph reconstructed on indirect data (Data source: Autorità di bacino del fiume Arno, 1996 [2]), used to calibrate the model. The curve in red shows the flood hydrograph at the river cross section of Firenze Uffizi obtained as result of the HEC-HMS model.

6 ECONOMIC DAMAGES ESTIMATION

About 3000 *ha* of the town of Firenze was flooded after the exceptional rainfall of November 1966. Heavy were the damages to private and public buildings, schools, hospitals and to the transport and hydraulic infrastructures. In addition, the consequences on the artistic heritage were dramatic: 1500 works of art and 1.300.000 volumes of the National Library were damaged [4].

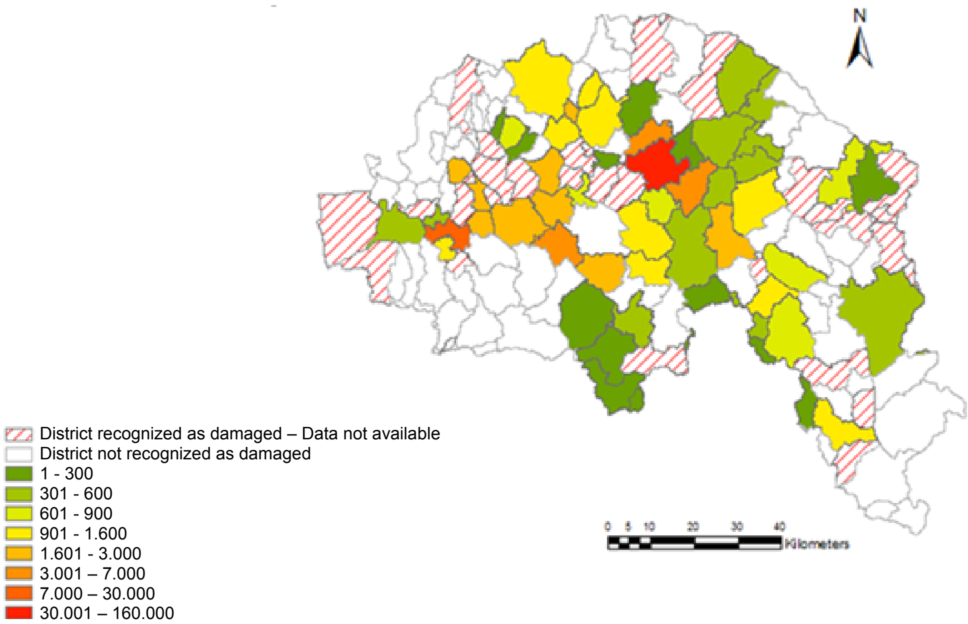


Figure 9. First estimate of economic damages, of the districts located in the Arno River basin after the flood of November 4th 1966, in millions of Lire, according the currency of the time

Considering some assessments made during the months following the event, an estimate of the economic damage suffered by the Tuscan districts after the flood of 1966 was carried out, using a historical survey approach. Selected data are provided by the AVI (Italian Vulnerable Areas) project [13], specific studies on this topic [3][8] and other official sources of the Tuscany Region [1].The result of the study is a map (Figure 9) in which the districts recognized as damaged by the flood are highlighted [7]. The damage framework highlights the seriousness of the event with a maximum loss, for Firenze town, of 150 billions of Lire, about two billions of current Euros, considering the ISTAT adjustment.

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ANALYTICAL SEISMIC VULNERABILITY ASSESSMENT OF REGIONAL HIGHWAY BRIDGES

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In this paper analytical method used for definition of vulnerability curves for road bridges characteristic for the Republic of Macedonia is presented. The method involves numerous nonlinear dynamic analyses of the considered structures. The obtained results were analyzed by statistic procedures from which the parameters characterizing the vulnerability curves were defined. The vulnerability curves represent the probability for achievement of a corresponding level of damage to the structure as a function of the corresponding intensity measure. To carry out the nonlinear dynamic analyses, the multiple stripe analysis approach was used. According to this approach, the analyses were carried out for a specific set of intensity levels and a certain number of time history sets. During each of these analyses, the parameters that have the greatest effect upon the stability of the considered structure were monitored. Once data were obtained from the analyses done for all time history sets and all intensity levels, the product of the binomial probabilities was computed for each level of intensity to obtain the probability for the corresponding dataset. The parameters of the vulnerability curves were obtained by maximizing the logarithm of the likelihood function using the lognormal cumulative distribution function. The used method for definition of the parameters that characterize the vulnerability curves does not require multiple observations for each intensity level and assumes independence of observations so that the overall likelihood is a product of the likelihoods at each IM level.

Keywords: Vulnerability assessment, Fragility curves, Bridge structure, NLTHA, Damage parameter

1. INTRODUCTION

Seismic risk assessment represents a key element in the formulation and development of strategies for mitigation and planning of earthquake consequences. In that respect, definition of vulnerability of existing bridge structures and bridge structures in the phase of design and construction, represents a critically important step in the process of vulnerability assessment.

To assess the seismic vulnerability of bridge structures in a certain region, in the ideal case, one should develop vulnerability curves for each structure taken separately based on its design parameters. This approach is very costly and hence practically inapplicable. Considering the fact that, within a certain region, many of the bridge structures are similar according to their characteristics, bridges can be generalized, i.e., they can be classified such that each bridge is classified into a certain group or type. In that case, seismic vulnerability is evaluated, i.e., vulnerability curves are generated for each type of bridge structues taken separately instead of each individual bridge. The details on the performed analyses of all types of bridges taken separately are given in [6].

1 DEFINITION OF A MODEL FOR ASSESSMENT OF DAMAGE LEVEL

The vulnerability function represents a probabilistic tool that is used to define damage occurring as a result of earthquake effect. It explicitly presents the probability for achieving or exceeding some ultimate level of damage under specific seismic excitation intensity.

Due to lack of realistic data on measured displacements, i.e., damages to bridge structures due to earthquakes in the territory of the Republic of Macedonia, the results obtained from nonlinear dynamic and static analyses of both systems of bridge structures typical for this region were used to define the criteria for definition of discrete states.

1.1 MODEL OF DAMAGE TO GIRDER BRIDGES

To define the discrete states of damage to bridge structures, the response of the bridge systems to seismic loads was, first of all, defined, i.e., the behavior of the bridge systems under seismic loads was monitored. To define the nonlinear behavior of the bridges and hence the capacity of all bridges, nonlinear static analyses were carried out whereat the bridge structure was loaded by a monotonously increasing force. The gradual increase of force led to progressive deterioration of structural elements and thus reduction of the structural stiffness. These analyses provided data on the bearing capacity, ductility and stiffness of the structural system and enabled monitoring of the development of material nonlinearities at different phases of loading of the structure enabling definition of damage levels. The response of the bridge structures was defined by numerous dynamic analyses in which the bridges were analyzed under the effect of real time histories of a scaled intensity.

The results from the nonlinear static and dynamic analyses represent a form of superstructure displacement - force relationships obtained for nonlinear static and nonlinear dynamic analyses of all four representative bridges. In these investigations, the relationships between the maximum displacement obtained from the performed nonlinear analysis of the superstructure U and the ultimate displacement of the bridge superstructure U_u (Eq. 1) were used to define the damage model.

$$DI = \frac{U}{U_u} \qquad (1)$$

This criterion was defined based on the main relationship: the seismic demand – capacity of the bridge structure as a complex structure consisting of a superstructure, central piers and bearings, including soil-structure interaction. The ultimate displacement U_u, (Eq. 1) represents the displacement of the superstructure representing the sum of the ultimate displacement of the pier and the ultimate displacement of the bearing in longitudinal and transverse direction, respectively.

When the seismic vulnerability of one type of bridges is defined, representative bridge structures are selected for analysis. Since they have different capacity, the index is to be normalized in accordance with the displacement index at the yield moment DI_y for each bridge according to its capacity. In that way, normalized damage indices (I) are obtained.

$$I = \frac{DU}{DI_y} \qquad (2)$$

For the purpose of differentiation of deformations, i.e., damages, three levels or limits that define the vulnerability of the bridge structure are differentiated. The first indicates the moment when yielding takes place. Until the yield moment, i.e., achievement of displacement U, minor deformations of the structure occur, i.e., the structure does not suffer any damage. At that moment, the damage index amounts to 1 (in accordance with Eq. 2). The second limit is set up in accordance with the assumption that the structure suffers larger deformations, i.e., the displacement exceeds the yield limit U_y, but not $2U_y$. The third level refers to displacements greater than $2U_y$, but smaller than the ultimate displacement U_u whereat the structure suffers large displacements, i.e., damages. For all displacements higher than U_u, it is adopted that the structure suffers failure. It should be mentioned however that, while defining the levels of deformations, i.e., damages, it is assumed that most of the displacements take place in the bearings. These ultimate values are shown in Table 1.

Table 1. Defined damage levels

	Level	Damages	Index
1	Negligible deformations	No damage	<1
2	Moderate deformations	Minor damage	1-2
3	Large deformations	Extensive damage	2-3.5
4	Failure	Failure	>3.5

2 STATISTIC ANALYSIS OF RESULTS OBTAINED FOR THE RESPONSE OF CHARACTERISTIC TYPES OF BRIDGE STRUCTURES

2.1. STATISTIC ANALYSIS OF RESULTS REFERRING TO RESPONSE OF GIRDER BRIDGES

Using the verified software package FELISA/3M [2], [4], [5], nonlinear dynamic analyses of representative girder bridge structures typical for the Republic of Macedonia were performed according to the finite element method. Each structure was analyzed by use of nine time histories scaled to 7 intensities (0.20g, 0.25g, 0.30g, 0.35g, 0.40g, 0.45g and 0.50g). Most of the analyses were carried out by a time step of 0.002 seconds. A total of 252 nonlinear analyses were performed.

To define the distributions of likelihood of occurrence of damage due to earthquakes of the same intensity expressed through the relationship between the maximum acceleration and the ground acceleration, the procedure proposed by Gumbel in 1934 [3] was used. According to this procedure, when there are N random values of the considered variable (in this case, the damage index), the first step is to arrange these according to size, starting from the lowest upwards. To each of such arranged values, there corresponds a cumulative probability that is defined by use of the following expression:

$$f_x(x) = P(X \geq x) = \frac{m}{N+1} \qquad (3)$$

where m represents the number of the damage index, while N is the total number of realizations of the variable. The value of the damage index and the corresponding defined cumulative probability define a point on the damage index – cumulative probability diagram. The same diagram also shows the relationship of the assumed distribution function of probability. The assumed probability distribution function is adopted as probability distribution function of damage occurrence. Fig. 1 shows the obtained values of the damage indices and the assumed probability distribution functions for all earthquake intensities and all four representative bridge structures in longitudinal direction. In all cases, a normal probability distribution is assumed. Its probability density function is presented by the following expression:

$$f_x(x) = \frac{1}{\sigma\sqrt{(2\pi)}} exp\left[-\frac{1}{2}\left(\frac{x-\mu}{\sigma}\right)^2\right] \qquad (4)$$

where σ and μ are parameters that characterize the normal distribution, whereat μ represents the mean value of the values of the damage index obtained from the conducted analyses, while σ represents the variation, i.e., the standard deviation of the same values.

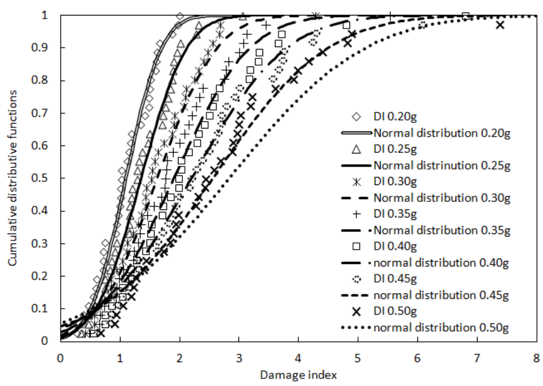
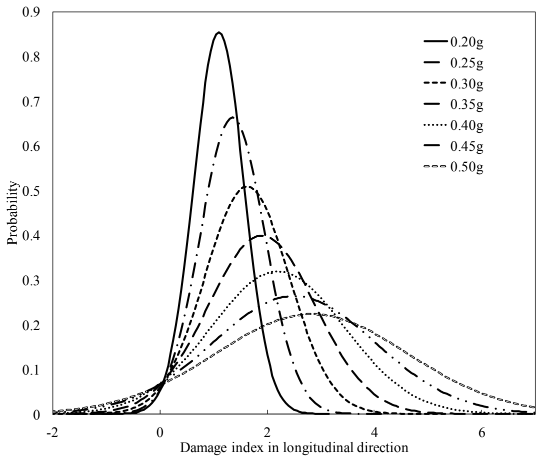


Figure 1. Cumulative distribution functions in longitudinal direction

The presented diagrams clearly show that the normal probability distribution correlates well with the probability distribution of occurrence of damage obtained from the analyses. The probability density functions of occurrence of damage in longitudinal and transverse direction are shown in Fig 2.



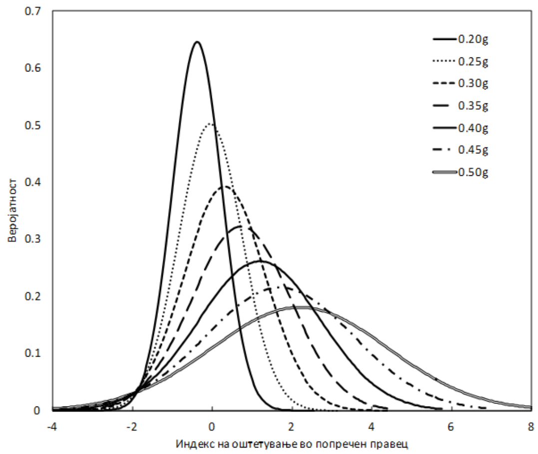


Figure 2. Likelihood density function for girder bridges in longitudinal (up) and transversal direction (down)

Table 2. Mean value and standard deviation under all intensity levels for girder bridges

	Longitudinal direction		Transversal direction	
	Mean value .σ	Standard deviation μ	Mean value σ	Standard deviation μ
0.20g	1.099	0.467	1.370	0.618
0.25g	1.354	0.601	1.700	0.795
0.30g	1.625	0.784	2.060	1.017
0.35g	1.903	0.999	2.473	1.238
0.40g	2.199	1.251	2.938	1.526
0.45g	2.54	1.512	2.436	1.852
0.50g	2.830	1.787	3.940	2.205

From the diagrams of the probability density functions, it is clear that the probability for achievement of a lower level of damage is greater in the case of low intensity levels. With the increase of the intensity level, the probability for occurrence of more extensive damage increases. Table 2 shows the characteristic values that describe the normal distribution for all earthquake intensities in longitudinal and transverse directions for girder bridges. There is a difference in the probability for occurrence of damage in longitudinal and transverse direction of the girder bridge structures.

2.2. METHODOLOGY FOR DEFINITION OF VULNERABILITY CURVES

To define the vulnerability curves, the multiple stripe analysis approach was used. It involves dynamic analyses of structures carried out for discrete levels of intensity, when different earthquake motions, i.e., time histories are scaled to a certain number of discrete levels. This approach is commonly used when selected earthquakes representative for a specific region are scaled for specific intensity levels since the critical parameter of the structure that is monitored, in this case the damage index, is changed under each intensity level. So, under the same intensity level of an earthquake, a certain structure will experience a certain level of damage, but it will experience another level of damage under another earthquake.

Fig. 3 displays the results for the damage index obtained from the nonlinear multiple stripe analyses of girder bridges in longitudinal direction.

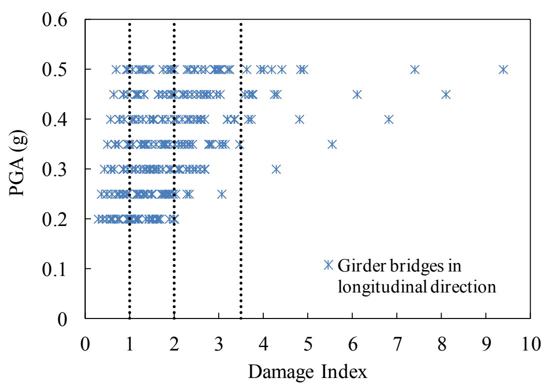


Figure 3. Results from multiple stripe analyses of girder bridges in longitudinal direction

The broken lines show the damage indices, i.e., the defined zones of deformations. The first zone A represents a zone of slight deformations where the damage index ranges between 0 and 1. Minor deformations occur when the damage index ranges between 1 and 2 (zone B); when the damage index ranges between 2 and 3.5, large deformations are at stake (zone C). If the damage index exceeds the value of 3.5, failure of the structure takes place (zone D). Since different earthquakes were used for each IM level, the IM level at which failure takes place cannot be exactly defined although it is expected that the true likelihood for occurrence of failure increases with the increase of IM.

For each intensity level, the analysis yields the number of damage levels due to the total number of earthquakes. Assuming that the damage level due to each earthquake does not depend on the results obtained for other earthquakes, the probability for achievement of the z_j level of damage due to nj earthquakes with $IM=x_j$ is given by the binomial distribution:

$$P\left(z_j\ level\ of\ damage\right)=\binom{n_j}{z_j}p_j^{z_j}\left(1-p_j\right)^{n_j-z_j} \qquad (5)$$

Where p_j is the probability that the earthquake with intensity IM will cause a certain level of damage. The objective is to identify the vulnerability function which will predict the likelihood p_j for each damage level. The maximum likelihood approach identifies the vulnerability functions for each damage level that provide the greatest likelihood of agreement of data from the nonlinear analyses of structures that would be carried out for other intensity levels with the corresponding damage level. When data from analyses are obtained for multiple levels of intensity, the product of the binomial likelihoods from equation 4 is computed for each damage level in order to obtain the likelihood for each input:

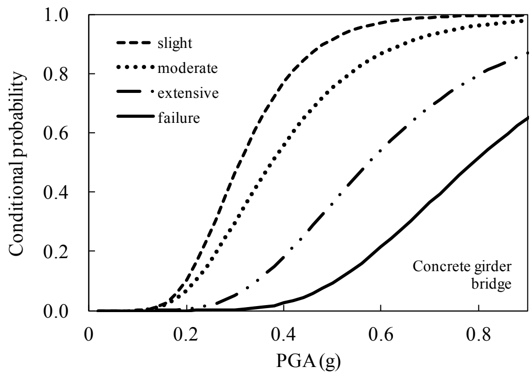
$$Likelihood=\prod_{j=1}^{m}\binom{n_j}{z_j}p_j^{z_j}\left(1-p_j\right)^{n_j-z_j} \qquad (6)$$

Where m is the number of levels of intensity, while Π is the product of all the levels (Baker [1]).

The parameters of the vulnerability function are obtained by maximization of this probabilistic function. It is equivalent and numerically easier to maximize the logarithm of the probabilistic function. This methodology predicts independence of the results so that the total probability is defined as a result of all probabilities for each intensity level. Also, it does not require multiple stripe analyses for each intensity level of interest. Therefore, this approach can be used to insert vulnerability curves without scaling of earthquakes. These can have the same intensity level.

2.3. VULNERABILITY CURVES FOR GIRDER BRIDGES

Using the methodology of multiple stripe analyses by application of the approach of maximizing the probabilistic function from the obtained number of analyses from which structures with a corresponding damage level were obtained, the parameters characterizing the vulnerability functions for all damage levels were defined. The values for the mean value of function θ and standard deviation β of ln IM (somewhere, it stands as dispersion of IM) were obtained for both orthogonal directions of the girder bridges and for each damage level. The vulnerability functions obtained for the girder bridges in both horizontal directions by use of this approach are shown in Fig. 4.



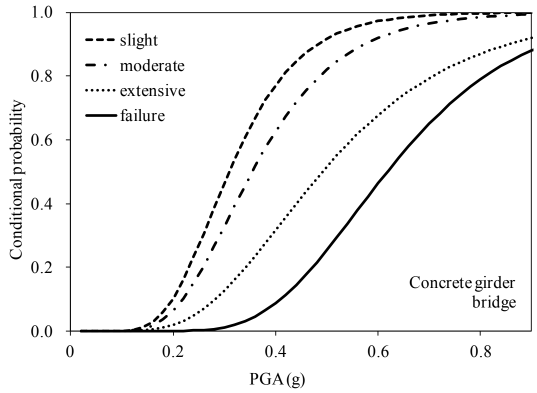


Figure 4. Vulnerability curves for a concrete girder bridge in longitudinal (up) and transverse direction (down)

CONCLUSION

Presented in this paper is a methodology for development of seismic vulnerability curves for road bridges. It has been used for typical reinforced-concrete bridges in the territory of the Republic of Macedonia. This methodology is practical and can be used for all types of bridge structures. A typical bridge structure is represented by a set of 3D models of representative bridges subjected to a suite of real ground motion records. The vulnerability curves show that the existing structures are with an acceptable level of vulnerability, i.e., for the designed level of intensity, there is a probability of 46% that the bridges will suffer minor damage, a probability of 30% that the bridges will suffer moderate damage and a probability of 5% that the structures will suffer extensive damage. There is almost no probability that failure of any of these structures will take place.

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EARTHQUAKES AND HEIGHT EFFECTS IN SOIL STRUCTURE INTERACTION PROBLEMS

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An earthquake force applied to a frame structure is through a soil media which plays an important role in behavior of the frame structures. It is also important to define the boundary conditions in the soil media in order not to let the reflections of wave propagation when simulating the soil structure interaction in numerical models. Last but not least is the effect of dampers in the structures which relate the behavior of the frame elements in such a way to optimize the response of the frame structures.

This paper points out the frame height subjected to soil structure interaction problems considering both the boundary effects in the soil media and the damping effects of the damper elements.

Keywords: soil structure interaction, seismic response, dampers

1 INTRODUCTION

In the structural analysis of an ordinary frame structure the soil structure interaction phenomenon is neglected and the dynamic response of the structure is evaluated under the assumption of a fixed based frame structure. However, during the earthquakes, there are big deformations that are seen in the soil and which are transferred to the foundation. This observation leads to questions of how the presence of structure alters the deformation in the soil and also how the structural response is altered by the presence of the supporting soil. This interaction is known as soil structure interaction and is important to be predicted when dealing with important structures.

The design loads that are considered in the soil structure interaction problems arise from the inertia forces developed in the frame structure and the soil deformations which are caused by the wave propagation in the soil medium.

In order to consider the soil effects in dynamic analysis it is useful to formulate the problem within the framework of finite element method. The complexity of the soil structure interaction problem makes the closed form of solution very complex thus the need for numerical methods arises. The source of the seismic force is not included in the finite element model which makes loading vector to have non zero values in the boundaries of the soil media. In order to deal in a successful manner with the boundaries in the soil media the presence of soil boundaries must be considered as special elements in the numerical simulations.

2 BOUNDARY CONDITIONS – INFINITE ELEMENTS

The dynamic analysis of soil media usually contains at least one side which prolongs to infinity. In order to simulate the infinite region in the numerical simulations silent, nonreflecting or transmitting boundaries are needed to be used. These boundary conditions are especially useful when considering the soil structure interaction problems. In the near field the effects of structural response are considered. In the near field the materials are influenced by state of stress, inelasticity, nonlinearity, volume changes etc. The far field is presented by boundary conditions. In the far field various procedures exist to model the infinite region. In this context one of the possible solutions for considering the far field is by using the infinite elements. The main advantage of the infinite elements is that the region is mapped in a successful way such that there is no need for extra work in calculations.

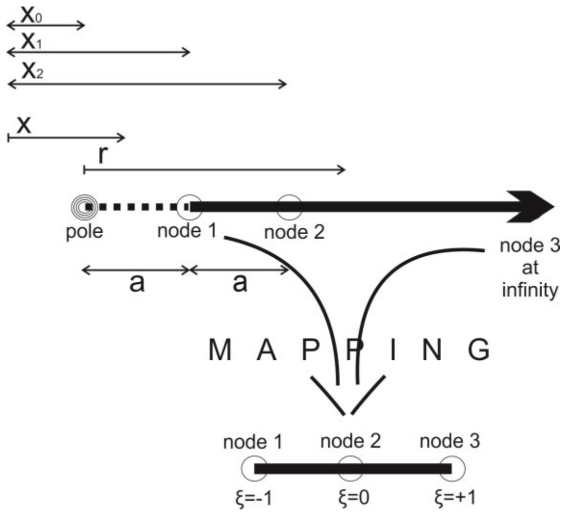

Figure1. Representation of the domain in infinite element

In this element, the field variables are approximated using the standard interpolation functions. In this way it is possible to obtain an expression with $1/r^n$ type decay of the field variables in the unlimited domain, where r is the distance from the 'pole' of the transformation to a point belonging to the infinite domain while n depends on the order of the shape functions used.

Some care should be taken in positioning the pole(s) of infinite elements, as the geometry and the field variable expansion depend on their position. Attention has also to be given to ensure the uniqueness of the mapping and the continuity of the solution between elements with common sides. The main advantage of the mapped infinite elements is the usage of the conventional Gauss-Legendre abscissa and weights. Infinite elements have first been developed by Bettess [1]. They have further been developed in frequency and time domain by different authors [2-4] The number and location of the nodes connecting finite and infinite elements must coincide to guarantee continuity condition between the elements. The main advantage of the proposed infinite elements is that the number of nodes on the infinite element allow coupling with finite elements with eight nodes which are used for displacement sensitive problems. Construction of element matrices is done by using the usual procedures as described in Bathe [5]. For the absorbing layer of the infinite element, the Lysmer-Kuhlmeyer approach [6] is used. By adding together the parts from each element constituent, the governing incremental equations for equilibrium in dynamic analysis are obtained. Time derivatives are approximated by the Newmark’s method and equilibrium iterations are used in each step.

3 DAMPERS

Mathematical modeling of dampers was done using combin14 element (Figure 2).

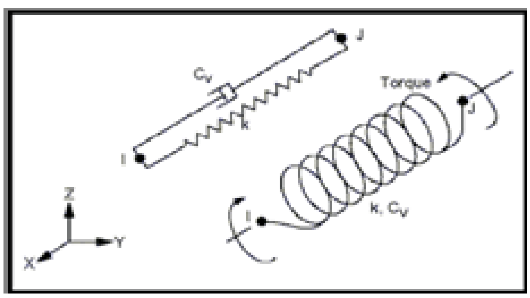

Figure 2. Analytical model for damper device

Mass of the damper, 60 kg, is added by using the appropriate mass element “mass21” in the software ANSYS [7]. The element works based on Kelvin Vought model and is defined by two nodes, a spring constant (k) and damping coefficients CV1 and CV2. The damping portion of the element contributes only damping coefficients to the structural damping matrix as given in work of authors [8-11]. Since the damper elements are pre-stressed with a force of 30 KN a preload in the spring as a compression is specified through an initial force input in the combin14 element. In the process of optimization for the damper elements the following characteristics have been used: stiffness of the spring K=1000kN/m, cv=35 kNs/m and pre-stress force F=30kN. This type of dampers achieves 10% added damping in the structure for reducing the response and improving the performance for earthquake excitation.

4 COUPLED SOIL STRUCTURE SYSTEM RESPONSE

In order to show the influence of storey numbers in frame elements considering soil structure interaction problems, a comparison of different height of structures are performed. In this direct time-domain method, the soil medium is modelled by two dimensional quadrilaterals using the finite element method. Similar soil-structure interaction problems have been studied in the works of other authors. In order to provide a complete insight, the soil side boundary of infinite elements is used. The frame structural elements are idealized as two dimensional elastic beam elements having three degrees of freedom at each node, translations in the nodal x and y directions and rotation about the nodal z axis. The behaviour of the frame structure is supposed to be elastic and has been modelled by using two parameters, the modulus of elasticity E=3.15x107 kPa and Poisson’s ration n=0.2. The bay length of the frame is taken to be 4.0 m, while the storey height is 3.0 m. The section of beams is 40 x 50 cm while that of the column is 50 x 50cm. For all frames, the beam and column sections, the floor masses and the number of bays are considered to be of concrete. The structures are modeled as one, four and eight-storey frame. The soil medium is presented as a two dimensional model composed of four layers resting on bedrock. In Table 1, the soil layers properties are tabulated in a way that the bottom layers are characterized by better soil characteristics.

Table 1 Soil properties

Number of layer	Thickness (m)	Unit weight (kN/m^3)	Shear velocity (m/s)
1	3	20	350
2	7	20	430
3	8	22	520
4	12	24	685

The soil is assumed to represent a linear-elastic material and is discretized by using eight nodded plane strain elements. The dynamic analysis has been performed by transient analysis using the step by step method. The proportional viscous damping matrix is taken to be proportional to mass and stiffness matrix (Rayleigh damping). Finite element modelling of the coupled soil-structure system is performed by use of the software ANSYS, as shown in Figure 3. The effect of soil-structure interaction is carried out by using the acceleration time history of the El Centro earthquake with a scaled peak ground acceleration of 0.30g. The moment transfer capability between the column and the footing is created by using a constraint equation where the rotation of the beam is transferred as force couples to the plane element as given in the ANSYS software manual.

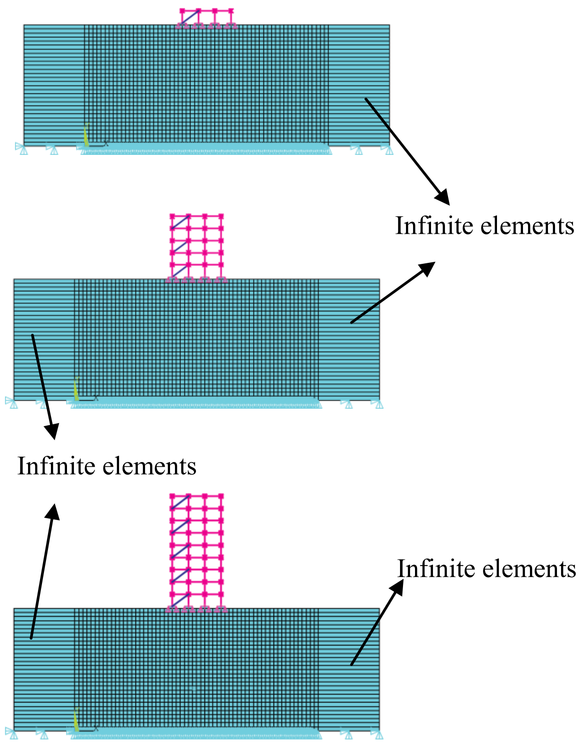

Figure 3. Frame element system with soil layers as foundation

In order to simulate the soil medium correctly side boundaries of infinite elements are used such that the wave propagation in the soil domain is simulated correctly.

The results obtained show interesting outcomes. First of all the structural moments obtained at the top of the structures are shown in the Fig. 4. It is quite interesting to see that the one storey structure has an effect of resonance due to the fact that the soil layers alone have a natural frequency of 10Hz.

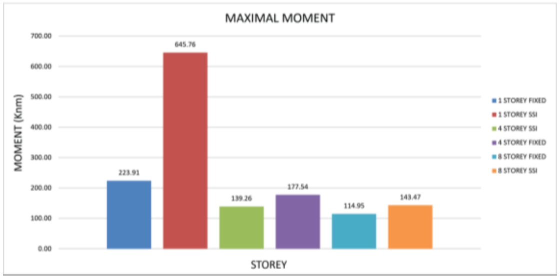

Figure 4. Variation of structural moments at the top of structures

On the other hand, the difference of structural moment results of four and eight storey structures show that the presence of soil layers does make difference in the results. On the other hand, the comparison of maximal acceleration as given in Fig. 5 clearly shows that the presence of soil layers increase the acceleration at the structural elements. This is a logical result since there is an expected amplification of the acceleration in the soil layers.

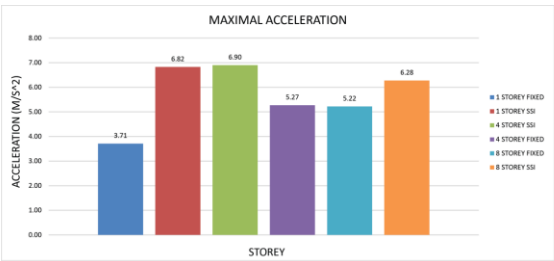

Figure 5. Variation of Structural response acceleration

The Figure 5 has an interesting point that the one storey structure has the biggest relative increase in the values. This is reasonable since the effect of resonance has the effects also in maximal acceleration. When considering the maximal displacements as given in Figure 6, it is quite interesting that the displacements are fairly smaller when the effect of soil structure interaction is considered.

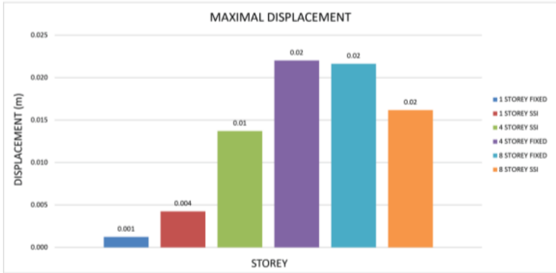

Figure 6. Variation of structural response displacement

Considering the above stated, it might be concluded that the soil structure interaction effects have both positive and negative effects on the results. Namely, the structural response of acceleration is increased when considering the SSI effects. On the other hand, the effects of soil layer presence decrease the maximum displacements at the structures. This holds mainly for bigger number of stories.

Since acceleration and displacement appear to be of importance in SSI problems, next the complete dynamic soil structure interaction time history analyses are performed.

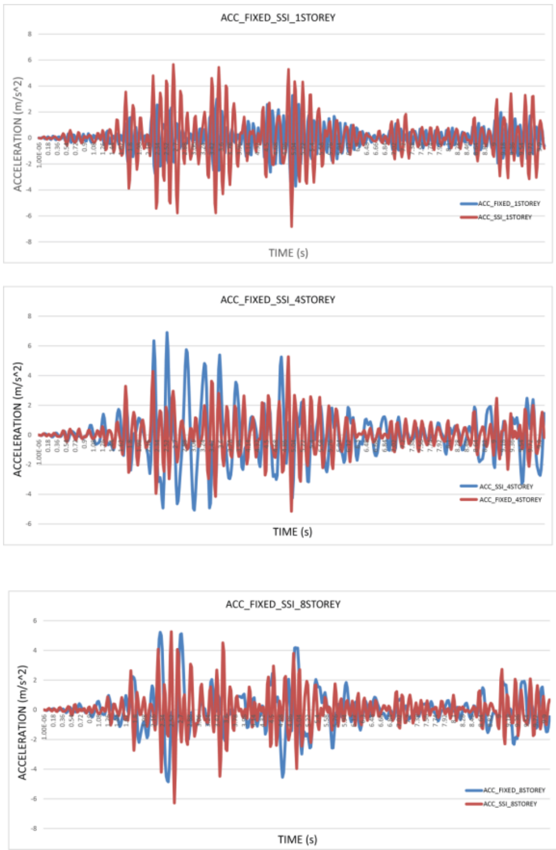

Figure 7. Comparisons of time history responses for acceleration for 1, 4 and 8 storey structure

As can be seen from Fig. 7, the acceleration time history is mostly influenced in the eight storey; the red lines which represent the SSI effects are dominant in the figure. This shows that the influence of SSI is crucial.

On the other hand, the surrounding filed of the domain is considered using the infinite elements which have the capability to simulate the infinite region very well. In numerical simulations ANSYS software is used where using its programmable features it is possible of programming new elements such as the infinite elements.

In order to make the comparison complete, different storey number for frames have been used. Evaluation of numerical analysis shows that one storey frame has small soil structure interaction influence. In contrast, when the storey number increases a more significant soil structure influence was observed.

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INFLUENCE OF FIRE COMPARTMENT POSITION ON BEHAVIOUR AND FIRE RESISTANCE OF RC FRAME

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This paper presents the behavior and the fire resistance of RC frame subjected to fire, in case of different fire scenarios. Four different fire compartment positions i.e. fire scenarios are analyzed and the internal forces, the deformations of the structure and the fire resistances are presented and compared. The temperature distribution within the structural elements and the structural response are obtained by the program SAFIR 2014. The numerical analyses indicate that the second floor fire compartments contribute to lowest fire resistance of the frame and highest deflections of the corresponding beams.

Keywords: reinforced concrete frame, fire scenario, fire resistance, temperature

1. INTRODUCTION

Besides satisfying structural design requirements for normal loads, such as dead and live loads, including seismic actions, buildings should also be designed to withstand a fire for a certain minimum duration as required for a desired level of performance or as it is defined in national legislations. In practice, this step is mainly neglected or only some types of buildings are controlled, but are not designed according to the fire safety criteria.

Defining the worst fire scenario is the main problem when fire design is considered. The reason for this is the fact that fire could occur at any location of the building and may cause different effects on structural elements and therefore different fire resistance will be reached.

Fires can cause significant damages to the structures, initiating reduction of the mechanical and changes in the thermal characteristics of the materials, decrease of strength and stiffness, large deformations, changes of internal forces etc [2]. All these effects are considered by the structural fire analysis specialized program SAFIR, used for the analysis presented in this paper.

2 NUMERICAL ANALYSIS OF RC FRAME

2.1 DESCRIPTION OF THE ANALIZED STRUCTURE

The three bay two storey RC frame, analyzed in this paper, is shown in Figure 1. The structure is made of concrete with compressive strength f_c=30 MPa and reinforcing bars with a yield strength f_y=400 MPa. The cross sections of all beams are 0.35x0.45 m^2 and the column sections are 0.40x0.40 m^2. Uniformly distributed load of 50 kN/m (including self weight) is applied on the beams (q/q_u≈0.6) and forces of 12 kN are applied in the beam to column joints of the first floor. Geometry, support conditions, reinforcement details and fire scenarios are shown in Figure 1. Depending on the fire scenario, beams are fire exposed on three sides (bottom, left and right side), when fire is assumed under the beam, or only on the top side, when fire is assumed over the beam. Fire scenarios are assumed to be only in one bay of the first or the second floor and from that reason the columns are heated only from one side.

2.2 FIRE SCENARIO I

The Fire scenario I assumes fire in the left first floor compartment and the fire resistance is t=197 min=3.28 hours (Figure 2b). Beam B1 is fire exposed at the bottom and the sides and columns C1 and C2 are heated from inside. The inner compartment parts of the cross sections of these 3 elements become hotter than the outer parts and tend to expand more than the outer parts. This cross sectional temperature difference causes the ends of the elements to tend to lift from the supports thus increasing the reactions. This action results in redistribution of moments. The negative end moments of beam B1 increases while the positive moment decreases and tends to become negative (Figure 2a and Figure 3). In this case the extreme values are at moment t=11 min. This moment depends on the fire scenario and of the cross section`s dimensions.

After this moment, the temperature penetrates deeper in the cross sections, the temperature difference between the top and the bottom

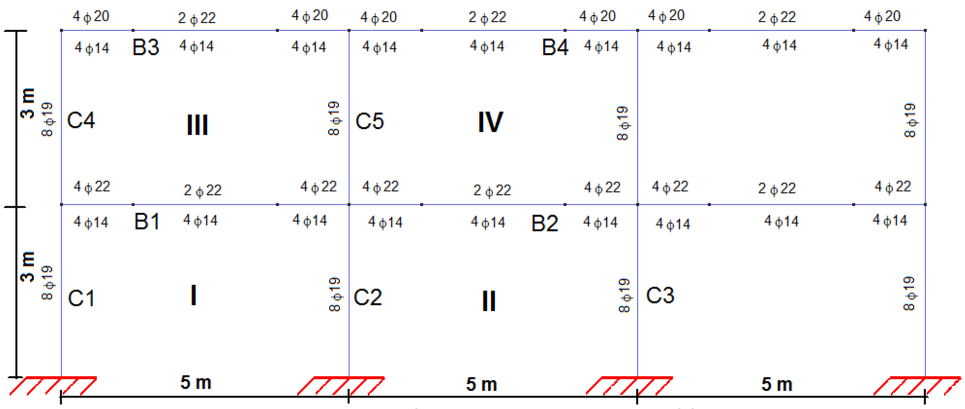

Figure 1. Frame geometry, reinforcement and position of fire scenarios

sides of the elements becomes smaller, and the bending moment diagram tends to return to the shape as for the moment t=0 sec, but it never reaches the initial shape.

In the first minutes of the fire large thermal expansion of beam B1 occurs, pushing the girder in adjacent bays, or outward. The longitudinal displacements of the joints are restrained by beam and column frame elements away from the fire zone, so compression force is developed in beam B1.

This axial force acts as a prestressing force and delays the moment when yielding of the top reinforcement will occur, so it has a positive effect on the fire resistance of the frame [1][3]. When fire occurs in the exterior bay and free expansion is partly allowed this force has smaller effect. This is the main reason why in the cases of fire scenarios in the middle bays (fire scenarios II and IV) the fire resistances are much higher than in cases when the fire is located in the outer bays (fire scenarios I and III). For the first fire scenario, the structural behavior is illustrated in Figure 2, where the moment redistribution at specific moments and the deformation of the frame at the moment of failure are presented.

Time redistribution of the bending moments at the ends of the beam B1 and the columns C1 and C4 are presented in Figure 3 and Figure 4.

During the first minutes of fire exposure, the tensile forces in the first floor beams, developed as a result of the mechanical loading, change in to compressive axial forces (Figure 5). At the same time, the second floor beams experience a change of the axial force from pressure to tension. All columns experience minor changes of axial forces.

At time t=21 min the bending moment at the top of column C1 has a highest value (140.2 kNm) and almost at the same moment the axial force in beam B1 reaches the highest value (71.63 kN) (Figure 4 and Figure 5).

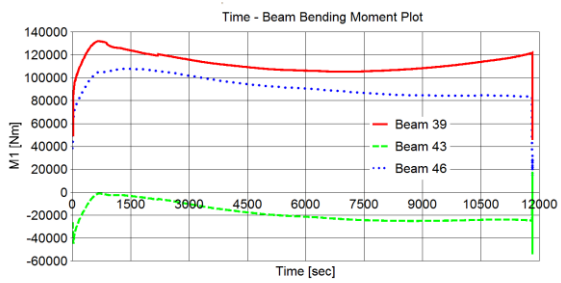

Figure 3. Time dependent bending moments at the ends and at the mid-span of beam B1, in case of fire scenario I

a)

b)

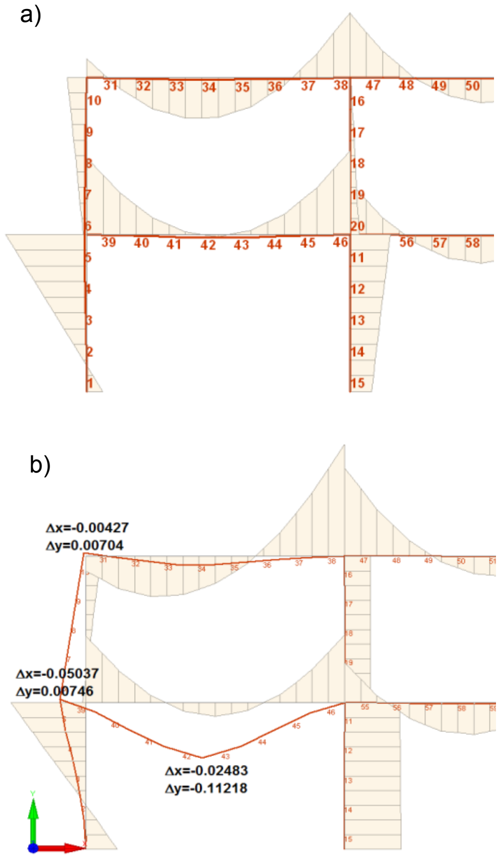

Figure 2. Bending moment diagrams a) at t=11 min b) at t=197 min (with failure mode), in case of fire scenario I

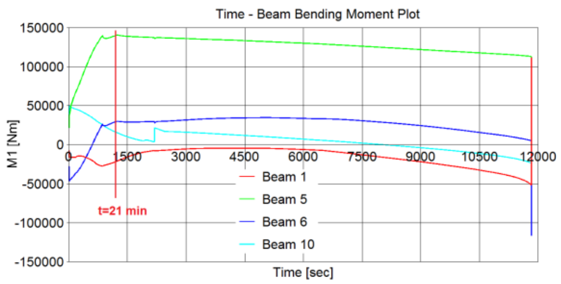

Figure 4. Time dependent bending moments in elements of columns C1 and C4, in case of fire scenario I

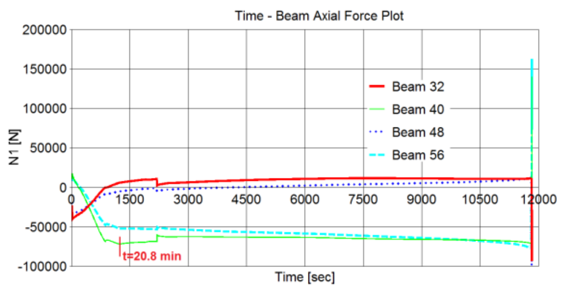

Figure 5. Time dependent axial forces in beams B1, B2, B3 and B4, in case of fire scenario I

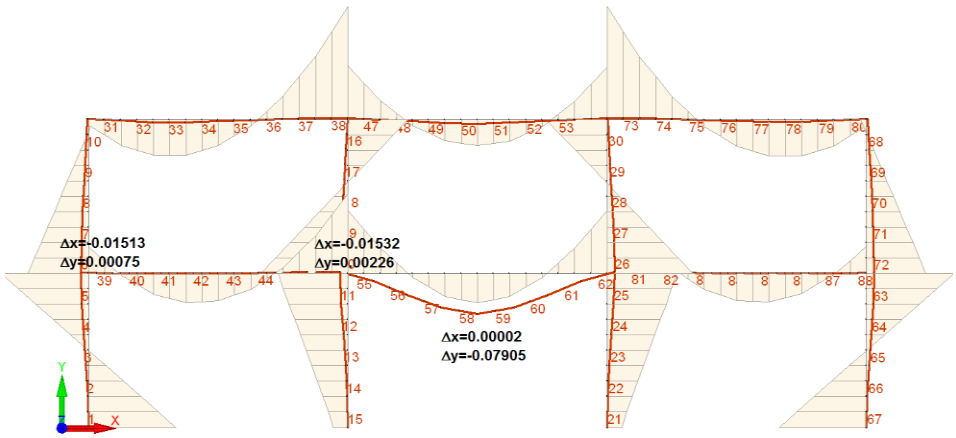

Figure 6. Bending moment diagram and deformation of the structure at the moment of failure (t=293 minutes), in case of fire scenario II

2.3 FIRE SCENARIO II

The Fire scenario II assumes fire in the central first floor compartment. The fire resistance is t=293 min =4.88 hours. In this case the thermal expansion of beam B2 is restrained by the surrounding unheated elements and significant compressive force is developed. The axial force has the highest value in comparison with the other fire scenarios and this is the main reason for the highest fire resistance of the frame. The vertical displacement at the mid-span of beam B2 reaches the lowest value (Figure 6).

2.4 FIRE SCENARIO III

The Fire scenario III considers fire in the left second floor compartment. The fire resistance is t=172.8 min=2.88 hours. In comparison with Fire scenario I, the fire resistance in this case is 12% less. The reason for this is the free outward thermal expansion of beam B3 and column C4. Consequently, the induced axial force has a lowest value.

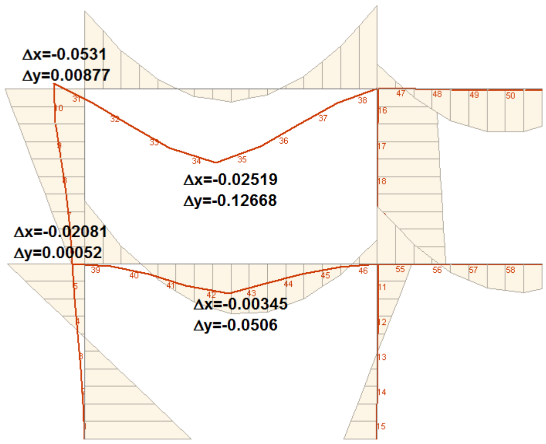

Figure 7. Bending moment diagram and deformation of the structure at the moment of failure (t=172.8 minutes), in case of fire scenario III

Horizontal displacements at the top of column C1 in Fire scenario I and column C4 in Fire scenario III are almost the same. The vertical displacement at the mid-span of beam B1 in Fire scenario I is smaller than of beam B3 in Fire scenario III (Figure 7). The reason is the partially restrained outward displacement of the beam to column connection in Fire scenario I, caused by the unheated column C4.

The bending moment at the top of column C1, as a result of the restrained thermal expansion caused by the unheated column C4, has a higher value in Fire scenario I (element 5, Figure 4) than in Fire scenario III (element 5, Figure 8). From the same reason the bending moment at the top of column C4 in Fire scenario III (element 10, Figure 8) is smaller than the corresponding one at the top of column C1 in Fire scenario I (element 5, Figure 4).

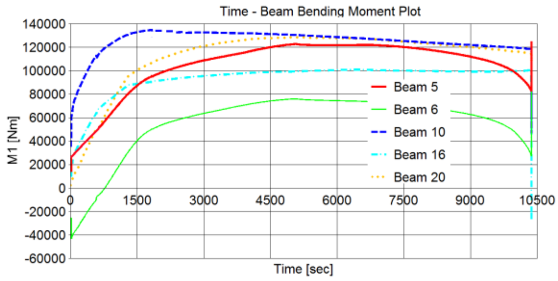

Figure 8. Time dependent bending moments in elements of columns C4, C5 and C1, in case of fire scenario III

2.5 FIRE SCENARIO IV

The Fire scenario IV assumes fire in the central second floor compartment. The fire resistance is t=209 min=3.48 hours. The bending moment redistribution at specific moments of the fire action and the deformation of the frame at the moment of failure are presented in Figure 9.

In this case the horizontal displacement at the top of column C5 is larger than the corresponding one at the top of column C2 in case of Fire scenario II. The reason is the higher stiffness of the beam to column joint as a result of the stiffer unheated column C5 in Fire scenario II. From the same reason the vertical displacement at mid-span of beam B4 is higher than the corresponding one of beam B2 in Fire scenario II. In the case of Fire scenario IV the beam B2 is exposed to fire only from the top side, therefore the temperature in the cross section is lower than in case of Fire scenario II. The effect of the non-uniform temperature distribution is the same as of the mechanical loading and the positive bending moment at the mid-span is increased, but the vertical displacement at the mid-span is less than in case of Fire scenario II because of the lower temperature in the cross section.

The time dependant bending moments for beams B4 and B2 are presented in Figure 10 and Figure 11, respectively. The reason for the bending moments redistribution is the same as in the previously described fire scenarios, but in this case the maximal values are reached at moment t=25 min.

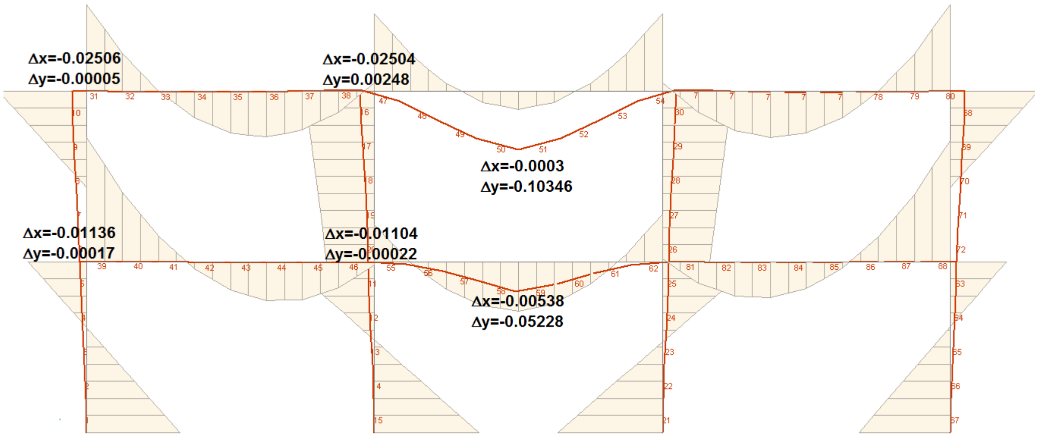

Figure 9. Bending moment diagram and deformation of structure at moment of failure (t=209 minutes), in case of fire scenario IV

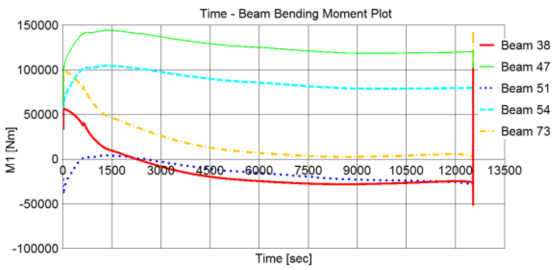

Figure 10. Time dependent bending moments at the ends and at the mid-span of the beam B4, in case of fire scenario IV

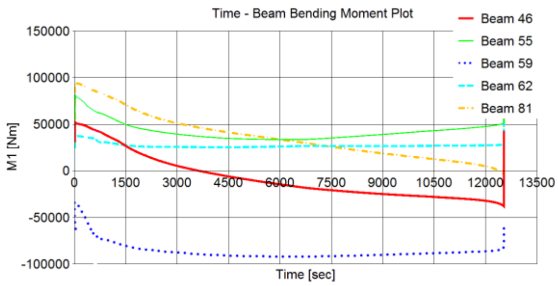

Figure 11. Time dependent bending moments in elements of beam B2, in case of fire scenario IV

The heated side of the cross-section of column C5 is in compression and the cold part of the section is in tension for the whole time of the fire exposure (Figure 12). The peripheral columns of the first floor are cold and behave like in normal conditions.

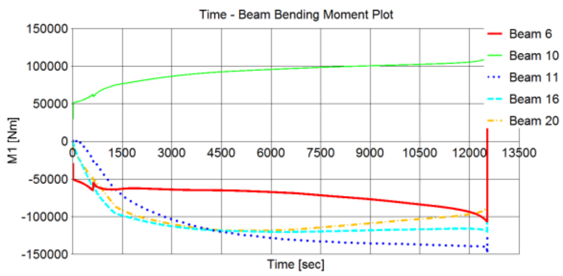

Figure 12. Time dependent bending moments in typical column elements of the frame, in case of fire scenario IV

3. CONCLUSION

Although some remarks and comparison of results were made in the previous sections of the paper, some general conclusions can be underlined here:

- When the fire compartment is considered to be the central bay only, the fire resistance is greater in comparison to other fire scenarios.
- For the same bay fire exposure, the higher the floor is the lower the fire resistance is.
- Because of the lower stiffness of the fire exposed external joints, the peripheral bay compartment fires lead to higher drifts in comparison to the central bay compartment fires and lower values for the fire resistance of the whole structure.

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