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Evaluation and Classification of the Dummy Loadings Regarding Biomechanical Protection Criteria during a Bus Rollover

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ABSTRACT: This paper presents simulation experiments and follow-up investigations of the passengers' biomechanical criteria during the rollover of various types of busses. The evaluation process focuses mainly on the reduction of the risk of injury. The partial aim is to indicate the influence of the safety restraint systems.

KEY WORDS: Bus rollover, bus crash, simulation, passive safety, biomechanical criteria, HIC, ECE R66.

1 INTRODUCTION

Today's road traffic is characterized by its high density. Under extreme traffic density conditions, the drivers' failure rate increases, and such mistakes often result in dangerous collisions. The most dangerous accidents, with respect to the passengers' injuries and fatalities, are accidents with buses, especially rollover. The Czech police statistics show that about 35 passengers have died and over 200 have been injured due to bus accidents since 2005.

For these reasons a high importance is paid to enhance the active and passive safety of the vehicles. These significant trends are particularly applicable to the development of the mass public transport means. Mandatory homologation of the new bus types leads to progressive increase of the passengers' safety, and the regulation ECE R66 was therefore introduced in 1986. This regulation forces the bus manufacturers to guarantee sufficient strength and stiffness of bus superstructures. Standing bus rolls over from a tilting platform, which is 80 cm above a flat ground. In fact no interior part of the vehicle may perforate into the so called residual space (a space inside the bus designed to keep passengers safe) during the rollover to prevent any excessive injury risk to the passengers. A couple of approaches to check the bus superstructure are described in ECE R66. Besides the real rollover test, which is very expensive, there is a possibility to substitute it with a numerical simulation.

TÜV SÜD Auto CZ is, among its other activities, traditionally focused on the evaluation of strength and stiffness of bus structures in the case of rollover. The evaluation is performed using real crash-tests and by numerical simulation methods. The experience acquired from these tests and virtual simulations make TÜV SÜD Auto CZ an appropriate subject to design passive safety solutions.

This paper is concerned with the investigation of the passengers' biomechanical criteria during the rollover of the bus. Three different types of buses were chosen to represent the construction classes (MI, MII, MIII). Another aim is to determine the influence of the restraint safety systems on the passengers' safety.

2 MATHEMATICAL MODELLING CONDITIONS

In the first phase the question of the simulation time and range to monitor the passenger behavior during the rollover was solved. The results were compared on a particular part of the bus chassis structure including the seats and dummy models. Another aim was to tune the model attributes in accordance with the passengers' behavior inside the vehicle.

In the next stage the typical representatives of the bus classes were chosen to perform the rollover simulation. Passengers' behavior in different types of buses was compared with each other. The bus types are represented by the coach, the suburban bus and the city bus.

Methodology of the rollover process has been acquired from ECE R66 regulation.
The simulation experiments were performed in PAM-CRASH/SAFE software by Mecas-ESI. The ARB Hybrid HIII 50% dummy model was used for all the simulations.

2.1 Determination of the simulation range

The partial aim of the project was to set the duration of the entire simulation of the bus rollover. To accomplish these conditions a model of the segment of the bus chassis with a seat and a belted dummy was created and two simulations were performed; a simulation with the roll-up phase and a simulation without the roll-up phase (the second simulation began in the point of unstable equilibrium position).

No significant difference in either the dummy or the chassis behavior was found from the comparison of the aforementioned simulations. The starting point of the following simulations was determined as the point of unstable equilibrium position of the vehicle (see Figure 1).

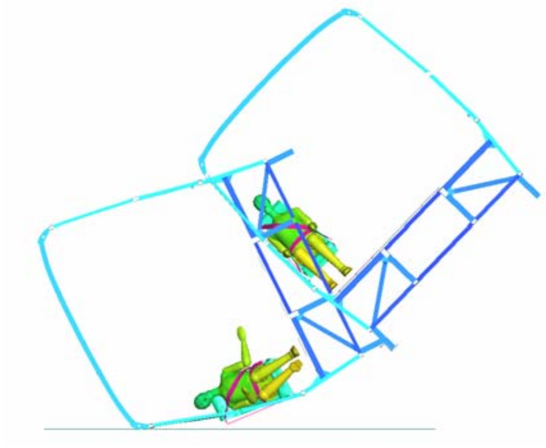

Figure 1: Rollover of the chassis segment – unstable equilibrium position

2.2 Validation with experimental simulation and verification

2.2.1 Rollover simulation of the chassis segment

The simulation of the rollover of the chassis segment is the final test, which are designed for tuning the material properties. The results of the final test are verified by rollover of the real segment prototype.

The material properties were verified using a comparison of the simulation and the real test of the chassis segment. The whole process of the real test was recorded with a high-speed camera. Selected important points were marked on the structure of the segment (see Figure 2).

The motion of these points was analyzed from the camera record and compared with the simulation. The residual deformation of the segment was measured after the rollover test. Figure 2 presents the results of the simulation compared to the high-speed camera record. The situation at 170 ms is displayed, which represents the highest deformation of the segment.

Figure 2: Highest deformation comparison (simulation vs. real test)

2.2.2 Tuning the model features with respect to the passenger behavior in the vehicle interior.

A couple of pre-simulations were performed in order to acquire the information of the dummy behavior when interacting with the vehicle interior, especially with the seat cushion. These pre-simulations help us to determine the seat cushion properties by the interaction between the dummy and the seat in the gravity field (see Figure 3). The material properties of the seat cushion were afterwards adjusted to correspond with the strength and stiffness of the real seat cushion on one hand, and to maintain the numerical stability of the simulations on the other hand.

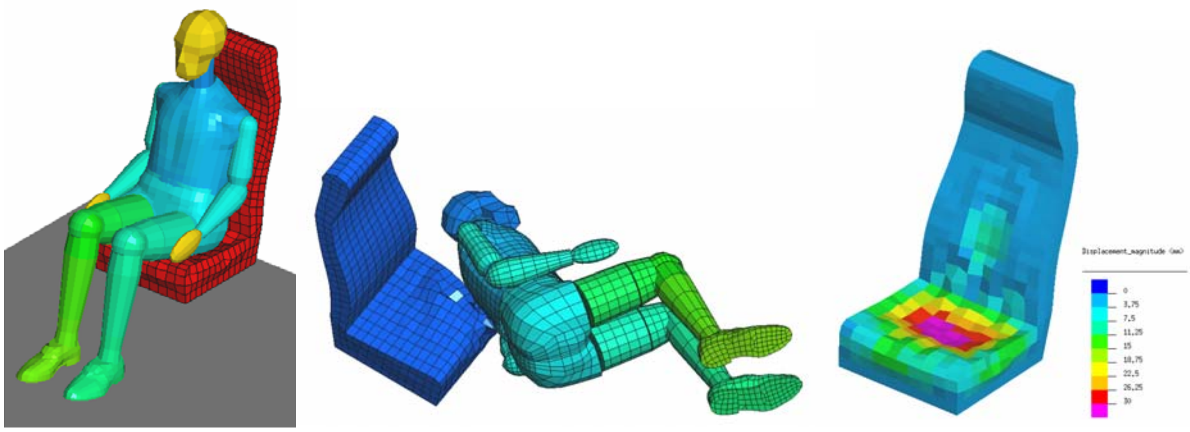

Figure 3: Properties tuning of the seat cushion

2.3 FE models of the representative bus classes

Three representative bus types were chosen to monitor the behavior of the passengers during the rollover:

- Coach (category M3/III) see Figure 4
- Suburban bus (category M3/II) see Figure 5
- City bus (lowdecker) (category M3/I) see Figure 6

The models were prepared for the virtual calculation according to the methodology described in ECE 66 regulation and EC 2001/85 directive. The Krupkowsky elastic-plastic material definition was used to describe the chassis material behavior.

Figure 4: FE model of the coach

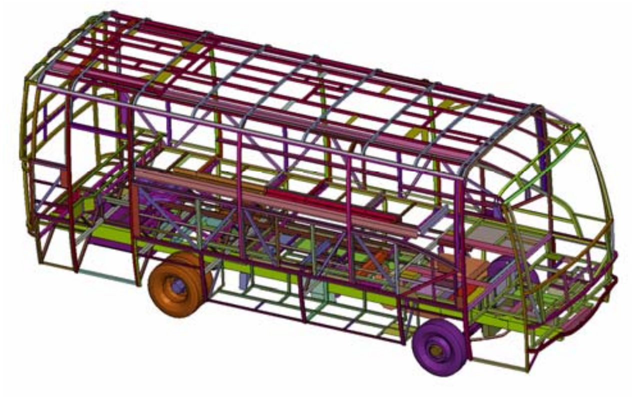

Figure 5: FE model of the suburban bus

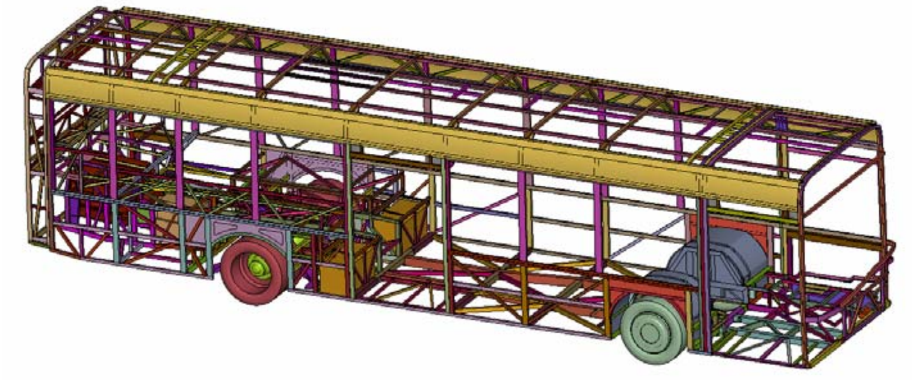

Figure 6: FE model of the city bus (lowdecker)

2.4 Calculation definition

The starting point of the simulations was set as the point of the unstable equilibrium position of the vehicle on the tilting platform. The passengers are seated in their initial position, i.e., their seats. The bus rolls over in the gravity field until it interacts with the ground, then the deformation of the bus structure follows. The total time of the simulation was set to 2500 ms. The aim of the simulations was to analyze the biomechanical loading of the passengers. The standard Head Injury Criterion (HIC15) is chosen to evaluate the head loading and the 3ms criteria are applied for thorax and pelvis of the dummies. HIC15 was evaluated from the CFC1000 filtered resultant acceleration of the dummy's head.

2.5 Passengers' arrangement

The passengers were arranged with the respect to the vehicle disposition, to monitoring possibilities of their behavior and to the simulation complexity. Therefore the passengers are seated in pairs or single in various rows, and also on both sides of the aisle to analyze the influence of the tilting direction. Figure 7 presents an example of the passenger arrangement.

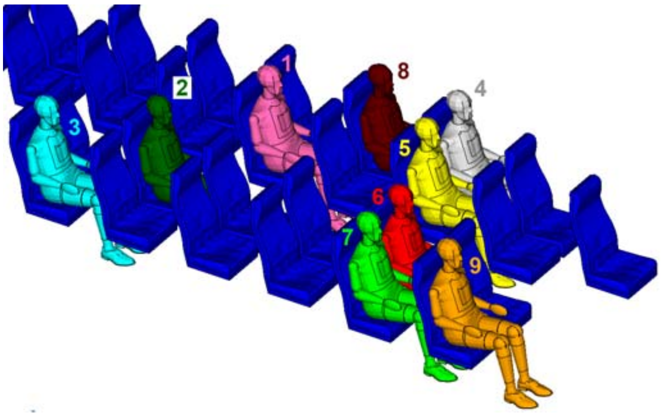

Figure 7: Seating of passengers in the coach

3 RESULTS

Figure 8 shows the final situation, after the interaction with the ground and the following deformation of the bus structure. The rotational impact velocity of the bus structure reaches the peak value of 5.1 m/s.

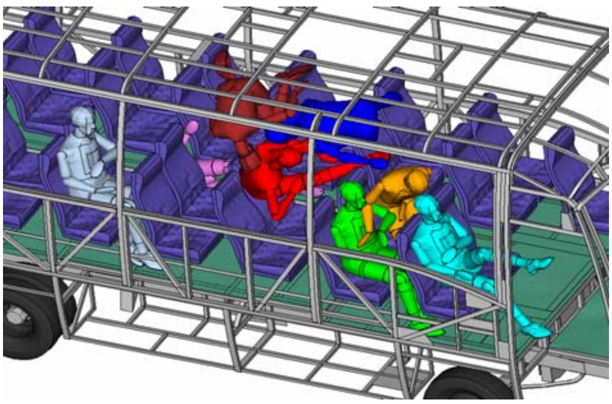

Figure 8: Rollover of the suburban bus

3.1 Data analysis

The maximum values of the biomechanical loading of the unbelted dummies are presented in Table 1, as well as the peak values of HIC15 and both thorax and pelvis 3ms criteria.

Tab.1 Maximum values of the biomechanical loading of the unbelted dummies

City bus (lowdecker)						
Dummy	**Head**		**Thorax**		**Pelvis**	
	HIC15	time [ms]	3 ms max. [g]	Time [ms]	3 ms max. [g]	time [ms]
Maximum values	**6 446**	791	**38**	1 831	**34**	2 221
	6 642	2 075	**36**	1 876	**35**	2 075
	Suburban bus					
Dummy	**Head**		**Thorax**		**Pelvis**	
	HIC15	time [ms]	3 ms max. [g]	time [ms]	3 ms max. [g]	time [ms]
Maximum values	**8 629**	2 291	**61**	2 298	**42**	2 299
	5607	1 993	**50**	467	**45**	2 291
	Coach					
Dummy	**Head**		**Thorax**		**Pelvis**	
	HIC15	time [ms]	3 ms max. [g]	time [ms]	3 ms max. [g]	time [ms]
Maximum values	**8 041**	411	**54**	998	**38**	588
	8 972	570	**115**	570	**50**	573

3.2 Range of the safety restraint systems application

The virtual calculations of the bus rollover with the two point safety belts application were performed with all the three bus types. The maximum axial force in the safety belts was included in the results. The calculations of the bus rollover with the three point safety belt application were performed with the coach and the suburban bus. Unlike the previous calculations the model city bus is not considered here. The manner in which the city buses are utilized, is not compatible with the application of the three point safety belts, with respect to high passenger flow rate requirement.

The aforementioned virtual calculations were completed with the calculation of the city bus rollover focusing on the biomechanical criteria evaluation of disabled persons.

3.3 Two point safety belts

Figure 9 presents the final positions of the rolled over city bus. The maximum values of the biomechanical loading of the two point belted dummies are presented in Table 2, as well as the peak values of HIC15 and both thorax and pelvis 3ms criteria.

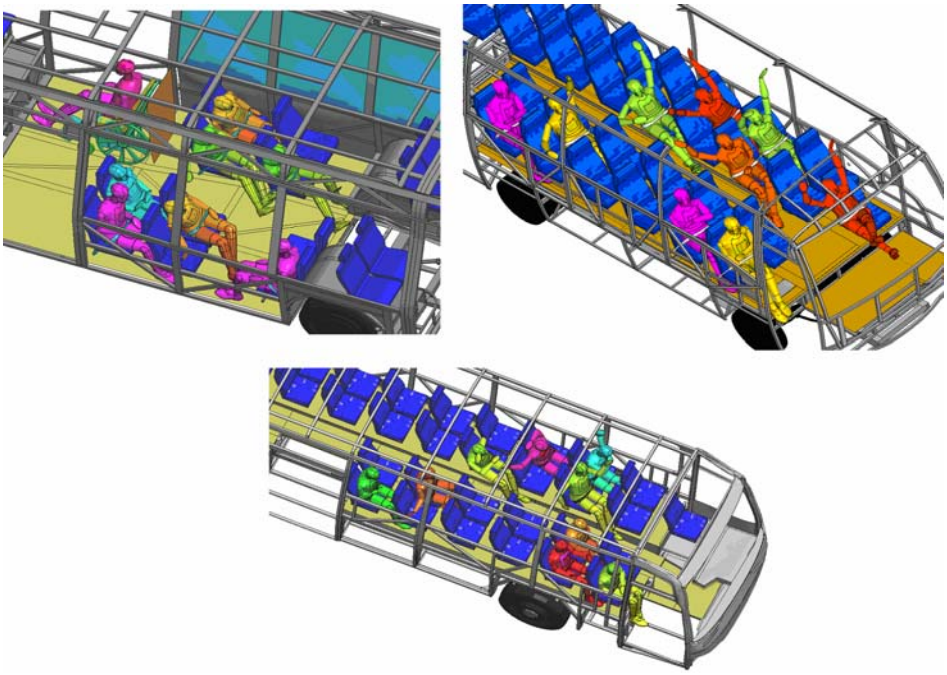

Figure 9: Rollover of the buses – two point safety belts

Tab. 2 Maximum values of the biomechanical loading of the dummies - two point safety belts

City bus								
Dummy	**Head**		**Thorax**		**Pelvis**		**Belts**	
	HIC15	time [ms]	3 ms max [g]	time [ms]	3 ms max [g]	time [ms]	force [kN]	time [ms]
Maximum values	**359**	1916	**48**	1917	**23**	1924	1,478	2010
	123	1920	**20**	2011	**13**	1954	1,04	2090
	5	1963	7	1999	8	2000	**2,569**	2010
	5	1964	10	1996	10	1997	**3,213**	2000
	Suburban bus							
Maximum values	**1692**	2092	15	2092	10	1964	0,818	1935
	4081	2350	**31**	2350	19	1920	**4,096**	1935
	186	1922	**22**	1923	**34**	1920	3,370	1945
	23	1894	10	1937	12	1957	**3,892**	1935
	Coach							
Maximum values	**3616**	1802	11	1706	14	1706	**5,760**	1760
	153	1802	24	1802	17	1802	**5,212**	1750
	52	1684	**38**	1685	**20**	2097	3,638	2120
	106	1685	**30**	1684	**20**	2077	1,479	2090

3.4 Three point safety belts

Figure 10 shows the final positions of the rolled over coach and suburban bus. The maximum values of the biomechanical loading of the three point belted dummies are presented in Table 3 as well as the peak values of HIC15 and both thorax and pelvis 3ms criteria.

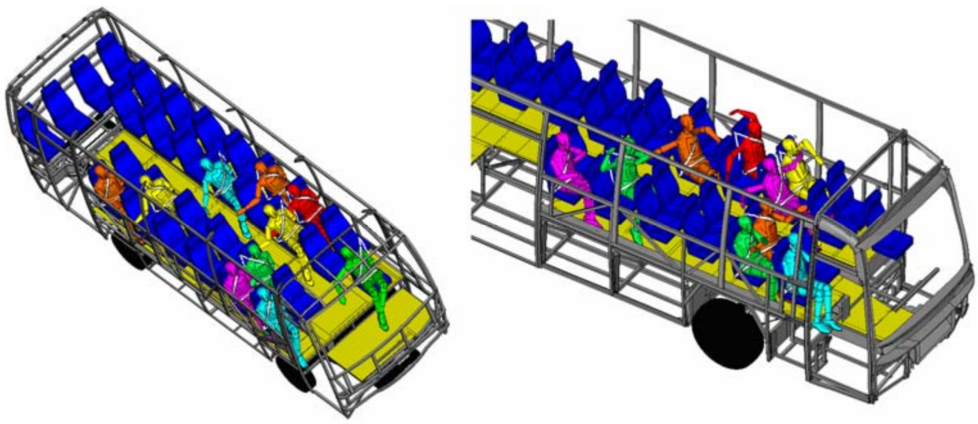

Figure 10: Rollover of the buses - three point safety belts

Tab. 3 – Max. values of the biomechanical loading of the dummies - three point safety belts

Suburban bus										
Dummy	**Head**		**Thorax**		**Pelvis**		**Belts**			
	HIC15	time [ms]	3 ms max [g]	time [ms]	3 ms max [g]	time [ms]	Shoulder part		Abdomen part	
							force [kN]	time [ms]	force [kN]	time [ms]
Maximum values	**208**	1651	**33**	1654	**20**	1726	2,135	2190	1,168	2130
	100	1656	**24**	1869	13	1867	4,408	2120	2,125	2310
	398	2087	18	2087	**20**	2070	2,544	2190	1,873	2110
	100	1665	15	1738	**21**	1699	3,754	1700	1,559	1730
	Coach									
Dummy	**Head**		**Thorax**		**Pelvis**		**Belts**			
	HIC15	time [ms]	3 ms max [g]	time [ms]	3 ms max [g]	time [ms]	Shoulder part		Abdomen part	
							force [kN]	time [ms]	force [kN]	time [ms]
Maximum values	**549**	1930	43	1986	31	1987	**4,231**	1940	2,038	1980
	675	1829	**88**	1912	**62**	1918	3,544	1990	**2,837**	2030
	490	1829	**58**	1829	**77**	1830	2,016	2090	1,835	1990

3.5 Disabled passengers

Figure 11 presents the final positions of the rolled over city bus. The view is focused on the disabled passenger, who was belted in the left part and not belted in the right part of Figure 11.

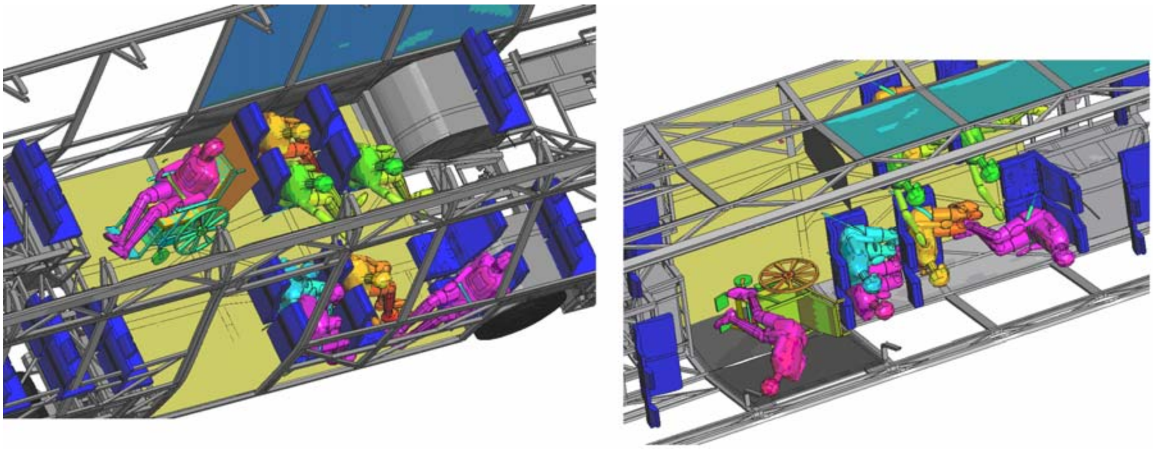

Figure 11: Rollover of the city bus focused on the disabled passenger (belted – unbelted)

The maximum values of the biomechanical loading of the disabled passenger are stated in Table 4. The peak value of the axial force is added to the relevant table.

Tab. 4 – Maximum values of the biomechanical loading of the disabled passenger.

Unbelted disabled passenger						
Dummy	**Head**		**Thorax**		**Pelvis**	
	HIC15	time [ms]	3 ms max [g]	time [ms]	3 ms max [g]	time [ms]
Maximum values	**154**	1165	**47**	1166	**22**	1165

	Belted disabled passenger							
Dummy	**Head**		**Thorax**		**Pelvis**		**Belt**	
	HIC15	time [ms]	3 ms max [g]	time [ms]	3 ms max [g]	time [ms]	force [kN]	time [ms]
Maximum values	**10**	1983	**6**	101	**11**	2008	**5,098**	2020

4 INTERPRETATION AND COMPARISON OF DUMMY LOADING VALUES

The comparison of the rotational impact velocities of the monitored buses indicates that the lower the position of Center of gravity (COG) of the vehicle is, the lower the rotational impact velocity also is. The difference in the rotational impact velocities is approximately 13%.

The values of the biomechanical criteria from the simulations with the belted and unbelted passengers were used to evaluate the influence of the restraint safety systems.

The behavior of the passengers during the rollover depends on the interior of the vehicle. High seat backrests in suburban bus and coach prevent passengers from moving across the vehicle. In the city bus the seats are placed lower and some of them may be turned backwards, unlike the seats in the suburban bus and coach. This allows the passengers to move through the entire width of the vehicle.

Up to 25% higher HIC15 values were found on the unbelted passengers in the coach and suburban bus compared to the city bus. These values are mainly affected by the aforementioned higher rotational impact velocity. The maximum HIC15 values were reached by the contact between the head of the dummy and the interior of the vehicle.

According to the assumption the fixation of the passengers into the seats prevents them from hardly any definable motion across the bus. Owing to the passengers' fixation, the risk of leaving the vehicle during the crash is minimized. Most of the dummies did not interact with either the surrounding seats, the interior equipment, or the other dummies. This fact had a major affect on the observed values of the biomechanical loading of the passengers, which are significantly lower compared to those values measured in the simulation with unbelted passengers.

5 CONCLUSIONS

The values of the biomechanical criteria from the simulations with the unbelted passengers are mainly affected by the position of COG of the vehicle.

The conclusions of the performed simulations refer to a fact that the biomechanical criteria in the case of fixation of the passengers are affected mainly by the COG position of the vehicle and the influence of the vehicle type usage is not that significant. The decisive factor, which particularly affects the biomechanical loading values, is the interaction of the dummy with the surrounding objects (other dummies, window pillars, inner coating, seats, etc.).

The analysis was completed with the peak axial forces in the belts. The values of the axial belt forces in the suburban bus and the coach are approximately 30% higher then the values measured in the belts applied in the city bus. It can be stated, that the axial belt force generally rises with the increasing position of COG. The highest value of the axial belt force was reached in the belt which held the disabled person (city bus), see Tab. 2 and 3.

The absolute values of the biomechanical criteria are affected by usage of the ARB HIII dummy model. This dummy model consists of rigid bodies connected by joint links. By the contact between the dummy and the vehicle chassis the recorded accelerations are notably higher then it would be in case of usage of the deformable dummy model. On the other hand, using the deformable dummy model would be excessively demanding for computation, as well as considerably more expensive.

6 ACKNOWLEGMENTS

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Interpretation of the Galileo Safety-of-Life Service by Means of Railway RAMS Terminology

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ABSTRACT: The Galileo Safety of Life (SoL) - Level A service has been designed mainly to support aeronautical operations ranging from en-route up to approach with vertical guidance (APV II). This is why the Galileo SoL Service has been specified by means of quality criteria coming from ICAO's (International Civil Aviation Organization) Required Navigation Performance (RNP) concept, i.e., in terms of accuracy, integrity, continuity, and availability. However, Galileo SoL – Level A service is also intended for railway signalling which is based on a different safety philosophy. The objective of this paper is to describe links among the Galileo quality criteria and the quality attributes of railway signalling systems (RAMS), according to the CENELEC standards EN 50126, EN 50129 and EN IEC 61508. This paper provides a basic theory and methodology for the employment of the GNSS quality attributes for practical design, validation and verification of railway safety systems based on GNSS. It has been shown that, in spite of the different safety philosophies used in aviation and railway safety systems, the Galileo SoL – Level A service can be described by means of RAMS terminology according to railway standards.

KEY WORDS: Galileo, RAMS, satellite navigation, safety, railway signalling.

1 MOTIVATION

Quality of railway signalling systems is described by dependability attributes such as reliability, availability, maintainability and safety (RAMS). A relationship among them is stated in CENELEC standards EN 50126 and EN 50129. In these standards a process for their specification and demonstration is also defined, which for "classical" signalling is quite clear. However, the situation becomes more complicated if a satellite navigation system is considered for railway signalling applications. A fixed ground infrastructure, which is usually distributed along the track and is the responsibility of a railway infrastructure manager, should be replaced by moving satellites with extensive ground infrastructure placed on more continents - all the responsibility of a multi-national operator. Moreover, the "core" of the signalling system, which provides safe train position determination, should be replaced by a system, which has not been primarily designed for railway signalling, but mainl for the aviation sector, where different safety principles are used.

The existing safety requirements for Signal-In-Space (SIS) of Global Navigation Satellite System (GNSS) were mainly driven by the needs of civil aviation. In 1993, the ICAO's Air Navigation Commission requested the All Weather Operations Panel (AWOP) to examine the possibility of an extension of the Required Navigation Performance (RNP) concept, which was originally intended for en-route operations, to include approach, landing and departure operations. It was proposed to include the following main GNSS quality criteria: a) accuracy, b) integrity, c) continuity, and d) availability.

A mutual relation among the GNSS quality measures and railway RAMS attributes was briefly outlined and explained in (Filip, 2006). From today's point of view, this brief explanation is insufficient, since preparation of railway applications requires a more comprehensive view. The aim of this paper is to provide a consistent background on which railway safety applications based on GNSS can be built. The work presented in this paper starts from the reliability of position determination and accuracy guarantee considerations. It is followed by a classification of GNSS failure modes that result from the GNSS quality measures definitions. Finally, the use of the GNSS failure modes within railway RAMS is described. The emphasis is not only focused on the significance of GNSS safety, but also on the analysis of GNSS availability and its impacts, which is important for the feasibility assessment of future GNSS based railway safety applications.

2 RELIABILITY OF POSITION DETERMINATION VERSUS SAFETY

Reliability R(t) of correct position determination by GNSS, including the correct function of its diagnostics, is outlined in Fig. 1(a) by means of Venn diagram. The position is correct when position error (PE) is maintained within a user defined alert limit (AL), i.e. PE ≤ AL.

(a) (b)

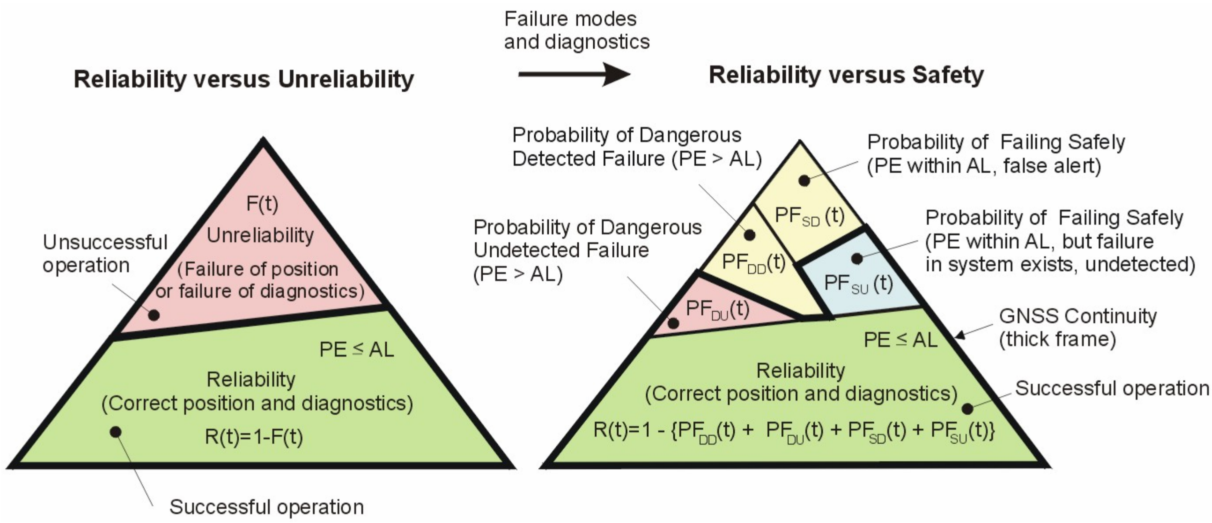


Figure 1: The relation between reliability and safety of position determination: (a) Reliability vs. unreliability, (b) Reliability vs. safety when diagnostics is implemented.

Reliability of position determination R(t) is a measure of success and it is a function of operation time interval (0, t). Unreliability F(t) of provided service or function is a measure of failure in time interval (0, t). It represents PE exceeding AL or/and a diagnostic failure. Unreliability F(t) is one complement of R(t).2.1 Failure modes

2.1 Failure modes

If failure modes are considered, unreliability of position determination function F(t) can be expressed as F(t)= $\{PF_D\ (t) + PF_S\ (t)\}$, and reliability as R(t)= 1- $\{PF_D\ (t) + PF_S\ (t)\}$. Probability of failing dangerously PF_D (t) represents the probability in time interval (0, t) that position error PE exceeds the alert limit AL, i.e. PE > AL. Probability of failing safely PF_S (t) represents the probability that PE ≤ AL. In this case, the output position from GNSS system does not influence the safety of the entire system. Nevertheless, there is a failure in diagnostics of the GNSS system or/and in position determination, which should be considered in dependability analysis.

2.2 Failure detection

Implementation of failure detection mechanisms can improve both safety and reliability. Subsequent refinement of failure modes will help to clarify the exact meaning of GNSS integrity and continuity risks, and it will help to find a way to describe them by means of railway RAMS terms, according to EN 50126. A relation among the reliability of position determination R(t) and the probabilities of failure modes is outlined in Fig. 1(b).

Probability of failing safely detected PF_{SD} (t) represents a probability that PE ≤ AL and that an alert is raised due to a failure of diagnostics. A false alert is then announced. The probability of failing safely undetected PF_{SU} (t) represents the probability of a non-critical failure when PE ≤ AL, but no failure is announced by built-in diagnostics. In this case, a safe failure in the system exists, but the user does not know about it. It can be revealed by an independent diagnostics based on physically diverse sensors, but this is outside the scope of this paper.

If PE exceeds AL and this hazardous state is detected, then it is a dangerous detected failure (true alert) and is represented by the probability of failing dangerously detected PF_{DD}(t). It is a part of the GNSS continuity risk CR(t), as will be shown below. The dangerous detected failure mode can be converted to the fail-safe state.

If PE exceeds AL without detection, it is a dangerous undetected failure, the so-called GNSS integrity risk, and it is described by the probability of failing dangerously undetected PF_{DU} (t). This state is the most feared failure in the system.

3 CONTINUITY

The purpose of continuity is to guarantee that the service of a navigation system or position determination function will not be interrupted when it is really needed. Therefore, the continuity requirement is defined for the most critical phase (a very short time interval, e.g. 15s) of a safety operation. Continuity C(t) approximately means reliability that a system works within the specifications (desired accuracy and integrity is provided) within a stated period of time interval (0, t). It is different from integrity, which means correctness of information. Loss of Signal-In-Space (SIS) continuity is caused by unscheduled interruptions due to internal fault detection mechanisms, and not by shadowing objects along track, which are well predictable. The exact difference between reliability and continuity is explained in the following paragraph.

3.1 Probabilistic description of continuity

The probability of correct continuous position determination (PE ≤ AL) by GNSS system under absence of any failure is described by reliability R(t) and is represented by the base segment in the pyramid diagram of Fig. 1(b). Unreliable position (unacceptable PE or/and failure in diagnostics) is described by failure modes which are represented by probability segments PF_{SD} (t), PF_{SU} (t), PF_{DD} (t), PF_{DU} (t) at the top of the pyramid.

The question is "under what conditions is the continuous provision of accuracy provided by the GNSS system?”. It is evident that it is not in a case of failure detection and notification. Therefore, probability PF_{SD} (t), i.e., probability of false alert, and PF_{DD} (t), i.e., probability of true alert, describe the probability of the interruption of the position determination function and belong to continuity risk (CR), see Fig. 1(b). Both undetected failure modes, represented by probabilities PF_{DU} (t) and PF_{SU} (t), exist during position determination since the user does not know them.

The probability of a hypothetical continuous provision of correct position ($PE \leq AL$) is equal to R(t) + PF_{SU} (t). The total probability of a continuous provision of position with acceptable integrity risk (possibly incorrect position due to a latent failure) is R(t) + PF_{SU} (t) ++ PF_{DU} (t), see Fig. 1(b). It is obvious that both undetected failures (SU, DU) can be revealed by a diagnostics based on physically diverse sensors.

3.2 Relation between reliability, integrity and continuity risks

Continuity is assured when a GNSS receiver provides, during operation: (1) navigation accuracy, and (2) accuracy guarantee, i.e., integrity. Accuracy is the only standalone parameter. The other GNSS quality criteria are accuracy dependent, as shown in Fig. 2(a). Continuity Risk (CR) is the complement of C(t), i.e., CR(t)=1 - C(t).

However, real systems can stop providing accuracy or integrity independently. In case of dangerous detected failure PF_{DD} (t) accuracy can be lost ($PE > AL$) while integrity (timely warning) is provided. Alternatively, in case of safe undetected failure PF_{SU} (t) accuracy is provided ($PE \leq AL$), even if the ability to provide timely warnings is lost. In this case, the user considers that system’s integrity is OK, since he receives integrity flags.

Therefore, it is necessary to distinguish three kinds of continuity: (1) Continuity of Accuracy, (2) Continuity of Integrity of Accuracy, and (1+2) Continuity of Service /Function. In the case of safety applications of GNSS, (1+2) continuity should be considered - see Fig. 2(a). Figure 2(b) outlines a relation among (1+2) continuity risk, reliability of position determination during the desired time interval and other failure modes.

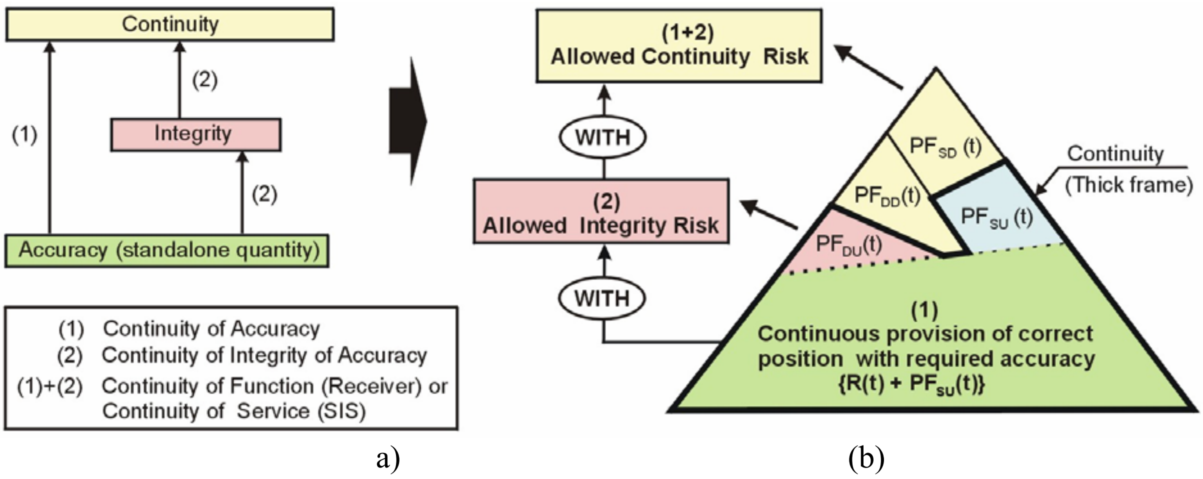


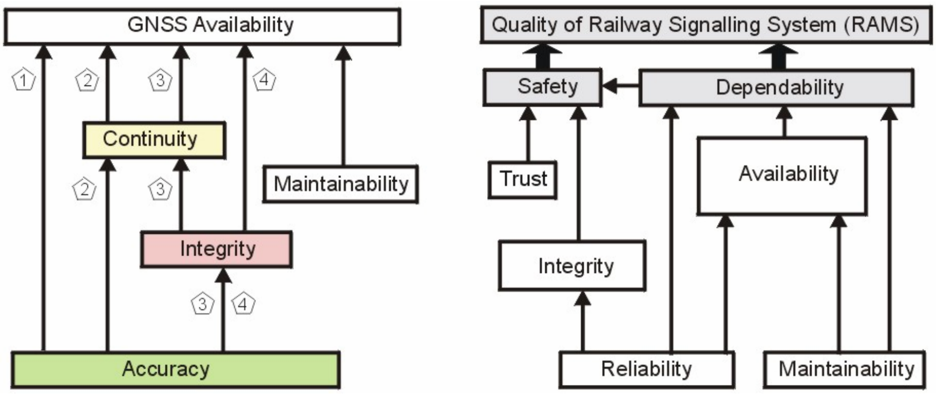
Figure 2: The relation among: (a) GNSS continuity, integrity, and accuracy, (b) Integrity and continuity risks and probabilities of failure modes.

Continuity C(t) in this figure includes R(t), $PF_{SU}(t)$ and acceptable GNSS integrity risk $PF_{DU}(t)$. Note that if once a quality of GNSS system is described in terms of failure mode probabilities, then integrity and continuity should be expressed in terms of risks IR(t), CR(t).

Dangerous detected failures (true alert) related to continuity risk are not as dangerous as undetected failures described by GNSS integrity risk. They can be converted to a fail-safe state. For example, a train can be stopped. However, it should be done only in an extreme case, when no other possibility exists. Relatively frequent interruptions of Galileo Signal-In-Space SoL Level A Service (MTBF=521 hours) can be substituted by a relative position determination, by means of additional physically diverse sensors (Filip, 2007).

4 GNSS AVAILABILITY VERSUS QUALITY OF SIGNALLING SYSTEM

The GNSS system is available if services of the system are within the required limits. That is, the requirements for accuracy, integrity and continuity of service/function are met. The mutual relation among the GNSS quality measures is outlined in Fig. 3(a).

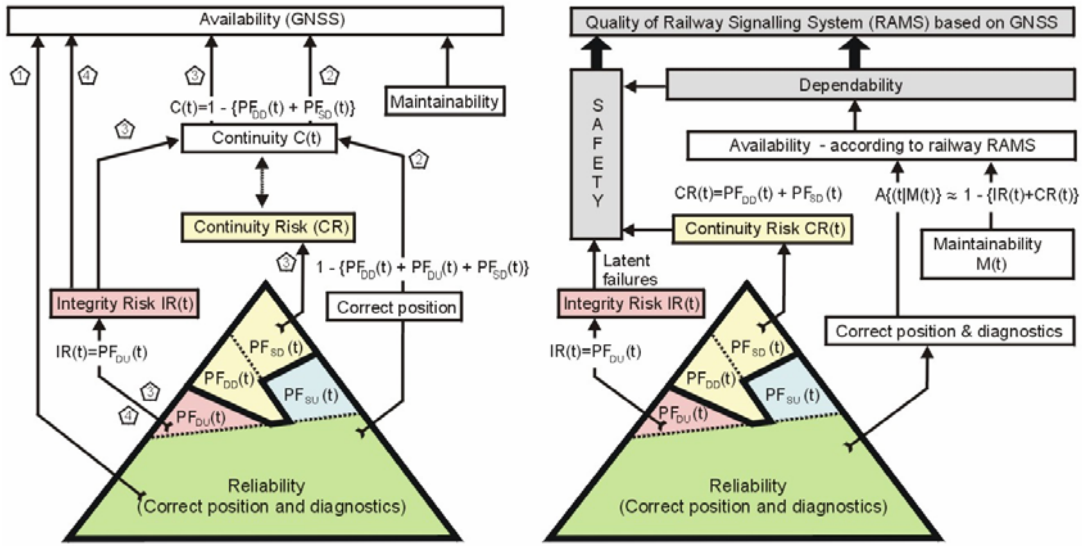


(a) (b)

Figure 3: The relation among: (a) GNSS availability, continuity, integrity, and accuracy, (b) Quality attributes of railway signalling system.

Since GNSS continuity and integrity are not standalone quality measures, as they depend on accuracy, then availability is guaranteed if the following conditions are simultaneously satisfied: (1) availability of accuracy, (2) availability of continuity of accuracy, (3) availability of continuity of integrity of accuracy, and (4) availability of integrity of accuracy – see Fig. 3(a). GNSS availability can be described by means of failure modes as shown in Fig. 4(a). Continuity C(t) consists of both unrevealed-by-diagnostics failure modes $PF_{SU}(t)$ and $PF_{DU}(t)$, and reliability R(t).

If a GNSS system is correctly operating at a time *t*, service is available. No requirement for successful operation during a specific time interval is directly involved in GNSS availability. However, a condition of continuous successful operation within a specific time interval (*0, t*) is involved in a lower rung, in the continuity requirement – see Fig. 3(a). Continuity is related to service or function interruptions within a specified (short) time interval. Continuity itself does not say anything about the percentage of time that a system should be in operation. A system can be accurate, have high integrity of accuracy and continuity but it can be down, e.g. 1 month per year. Maintainability must be assured. When maintainability is considered together with other quality measures, then we talk about the availability of continuity, availability of integrity, etc. GNSS availability at user level also implies that the GNSS receiver is able to predict the accuracy, integrity and continuity performance over the next critical operation period and that the predicted values must not exceed the specified values.



(a) (b)

Figure 4: Quality attributes of: (a) GNSS system, and (b) Railway signalling based on GNSS.

4.1 Quality attributes of GNSS based railway signalling

A relationship between the main quality attributes of railway signalling system (RAMS) is outlined in Fig. 3(b). As results from the "railway" definition of availability (EN 50126), a system is available if it is successful at a time *t*. In case of a location determination unit, successful operation means that, for example, a desired accuracy is met (PE $\leq$ AL) and diagnostics work correctly.

Availability (EN 50126) is a combination of reliability and maintainability. Trust of the provided accuracy is described separately from availability by means of integrity requirement - see Fig. 3(b). Safety requirements are not included in railway availability. In relation to railway safety related systems, we usually talk about availability (dependability) and safety.

On the other hand, GNSS safety requirements (i.e., integrity and continuity) are directly involved in GNSS availability as results from Fig. 3(a). GNSS system is available if also (among others) safety integrity requirement is assured.

The use of the GNSS quality criteria within railway RAMS is proposed in Fig. 4(b). It results from the analysis of GNSS integrity and continuity risks performed in the above parts 2 and 3. Instead of the notion of integrity in Fig. 3(b), the term GNSS integrity risk IR (t) is used in Fig. 4(b), which is one complement of integrity and expressed by means of PF_{DU} (t). In railway safety systems, a failure rate per hour shall be used instead of a probability per duration of operation for the purpose of a quantitative safety analysis. An approach to convert GNSS integrity risk to the dangerous failure rate per hour has been proposed in (Filip, 2007). GNSS continuity risk CR(t) includes both detected failure modes $PF_{DD}(t)$ and $PF_{SD}(t)$ as both have an impact on the safety of the system.

Availability A(t) according to EN 50126 depends on the correct position determination, correct function of diagnostics and maintainability M(t) of GNSS system. It can be written as A{t|M(t)}. Availability A{t|M(t)} can be evaluated by means of the probability of incorrect operations U{t|M(t)} under condition that maintainability M(t) is provided ($\forall$ t, M(t)=1).

The probability of incorrect operations of the GNSS system can be determined from the given integrity risk $IR(t)= PF_{DU}(t)$ and the continuity risk $CR(t)= PF_{DD}(t) + PF_{SD}(t)$ as $U\{(t|M(t)\}= PF_{DU}(t) + PF_{DD}(t) + PF_{SD}(t)$. The remaining probability of safe undetected failure mode $PF_{SU}(t)$ is, for the sake of simplicity, added to the reliability segment R(t). This simplification can be performed, since the correct position is provided. Then availability $A\{t|M(t)\}$ can be expressed as

$$A\{t|M(t)\}= 1- \{PF_D(t)+ PF_S(t)\} = 1- \{PF_{DU}(t) + PF_{DD}(t) + PF_{SD}(t) + PF_{SU}(t)\}= \\ = 1- \{IR(t) + CR(t) + PF_{SU}(t)\} \approx 1- \{IR(t) + CR(t)\}$$

In spite of the fact that GNSS service performance is defined by means of notions that come from the aviation sector, the railway sector can employ them, with respect to their specific meaning according to railway standards (see Fig. 5).

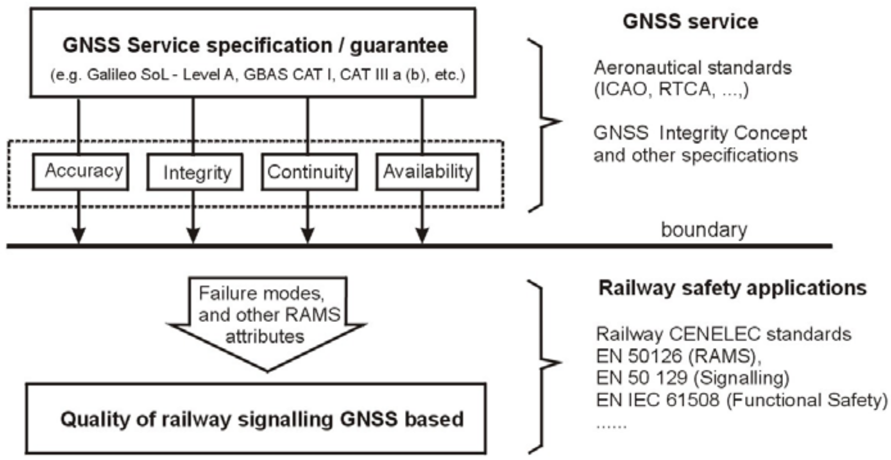


Figure 5: The relation between GNSS service specification and railway RAMS.

ICAO's Required Navigation Performance measures (continuity and integrity risks) can be converted to failure modes and used together with other GNSS RAMS (such as availability, MTTR after discontinuity event, etc.) for design and verification of GNSS based railway safety related system according to railway standards EN 50126 and EN 50129.

4.3 Availability for railway signalling

According to the Galileo SoL - Level A service specification (Galileo Integrity Concept, 2005) SIS should be available at 99.5% of time. It means that SIS for SoL Level A may not be available 438 hours per year. Note that possible SIS interruptions due to objects alongside the track and landscape profile are not included in this specification of availability. In some cases, due to SIS shadowing mainly on urban or mountain lines, conditions for utilization of the Galileo service can be much worse. A guarantee of EGNOS SIS service is much worse: it is not available at 95% of time, i.e., 438 hours per year, i.e., approximately 18 days.

Availability requirements for signalling equipment result from the safety and operational requirements for the entire railway transport system. For example, if a system based on GNSS should replace ERTMS/ETCS odometry, then unavailability less then 10^{-7} is required (ERTMS/ETCS RAMS, 1998). It means downtime for odometry subsystem should be less than 3.15 seconds per year**.** One can imagine how much augmentation of GNSS by additional sensors would be needed to achieve this very high railway availability target.

5 CONCLUSIONS

Knowledge of the exact relationship between GNSS quality measures and railway RAMS is important for the design and verification of railway safety related systems. In this paper, we have shown how to employ GNSS quality criteria according to railway RAMS. Analysis performed in this paper shows that GNSS continuity does not exactly correspond to reliability and what the difference between GNSS and railway availability is according to EN 50126. In spite of the different definitions and notions used for the description of GNSS quality criteria and railway RAMS, it is possible to find a relationship between them.

6 ACKNOWLEDGEMENT

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Limitation of Geosynthetics Usage on Road Subgrade

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ABSTRACT: There are several ways of geosynthetics functionality. Well-known mechanisms are functions of separating, filtrating, protecting and slope reinforcing. There are several publications describing the influence of geosynthetics on increasing the soft-soil bearing capacity and their positive role on final subbase deformation. However, this geosynthetics functionality has not been explained satisfactorily in detail yet. The research presented here deals with clarifying the possible geosynthetics functionality. Six kinds of geosynthetics of world renowned producers were selected for testing. The measurement was carried out in the Geotechnical Laboratory Testing Field (GLTF), which is a facility constructed for full-scale geotechnical measurements. The results of static plate tests show that the contribution of the geosynthetics to the bearing capacity increase is very limited. A significant increase appears only in the case of a very low bearing capacity subgrade covered by a 20 cm thick subbase layer reinforced by some geosynthetics. After the static part of the experiment the GLTF was equipped with a cyclic loader to simulate real traffic loading and tests for the evaluation of possible bearing capacity increase and deformation decrease due to geosynthetics usage were repeated.

KEY WORDS: Subgrade, pavement, geosynthetics, bearing capacity, deformation

1 INTRODUCTION

Although the reinforcing functionality of geosynthetics is described and used in connection with slopes reinforcement, there are many practical examples of geosynthetics usage as a reinforcing element of pavement, subbase or subgrade layers. At the same time, this application is put to laboratory, full-scale or real construction tests. The results and outputs of the tests are highly incomparable because of non-uniform test methodologies and some of them being purpose-built by geosynthetics producers. Maybe this could be the reason why the EU standards do not include this way of geosynthetics application.

Principles of the evaluation of the geosynthetics impact on bearing capacity are based either on the static loading of reinforced layers (Blumme et al., 2001), expressed by the relationship between loading and deformation (elastic modulus, deformation modulus or k-modulus, conversion to or from CBR), or on cyclic/dynamic loading done either by a circular plate (Perkins, 2002) or by repeated wheel running (Watn et al., 1996 and Jenner et al., 2002), evaluated by a time-based deformation (number of loading cycles or wheel running vs. layer deformation). Based on the time-based deformation the back-calculation methods were formulated (e.g., van Niekerk and van Gurp, 2002).

It is essential for each experiment to take into account the following parameters: material properties (soil, aggregate, geosynthetics, etc.), test condition (subsoil bearing capacity, layers compaction rate, etc.), geometrical parameters (dimensions, model similarity) and test arrangement (speed, range and frequency of loading, shape of the loading wave, etc.).

Unfortunately, many published papers are missing the mentioned parameters. This is a very serious cause for the incomparability of the published results. The tests are often based on "national" test methods and are valid only when using specific geosynthetics (e.g., Beckmann and Ruppert, 1994, Saathoff and Horstmann, 1999). There is a comprehensive review of sophisticated tests containing available tests parameters (Berg, 2000).

2 ENVIRONMENT, PARAMETERS AND RESULTS OF STATIC TESTS

The tests were carried out in the Geotechnical Laboratory Testing Field (GLTF). The GLTF is a facility available for full-scale geotechnical tests, which allows for the measuring in a laboratory of some geotechnical quantities that are otherwise usually measured in the field (e.g., plate test, dynamic loading test, penetration test, etc.) on various soils and soil layers for different compaction rate and for different water regimes.

The GLTF, see Figure 1, is about 10 meters long and consists of a concrete pit split by removable dividers into separated measuring (testing) spaces and a watering/dewatering drain channel, also separated by removable dividers. There is a drain layer placed on the bottom of each measuring space covered with a grate with a drainage geotextile (filter). Both the concrete pit and the drain channel are interconnected at their bottoms. A moveable frame can be slid in the longitudinal direction along a guide-way (rails) fastened on the top of the pit. The moveable frame serves for mounting or supporting the measuring equipment (plate test, CBR in situ test equipment, etc.) and can be blocked in both horizontal and vertical directions during testing.

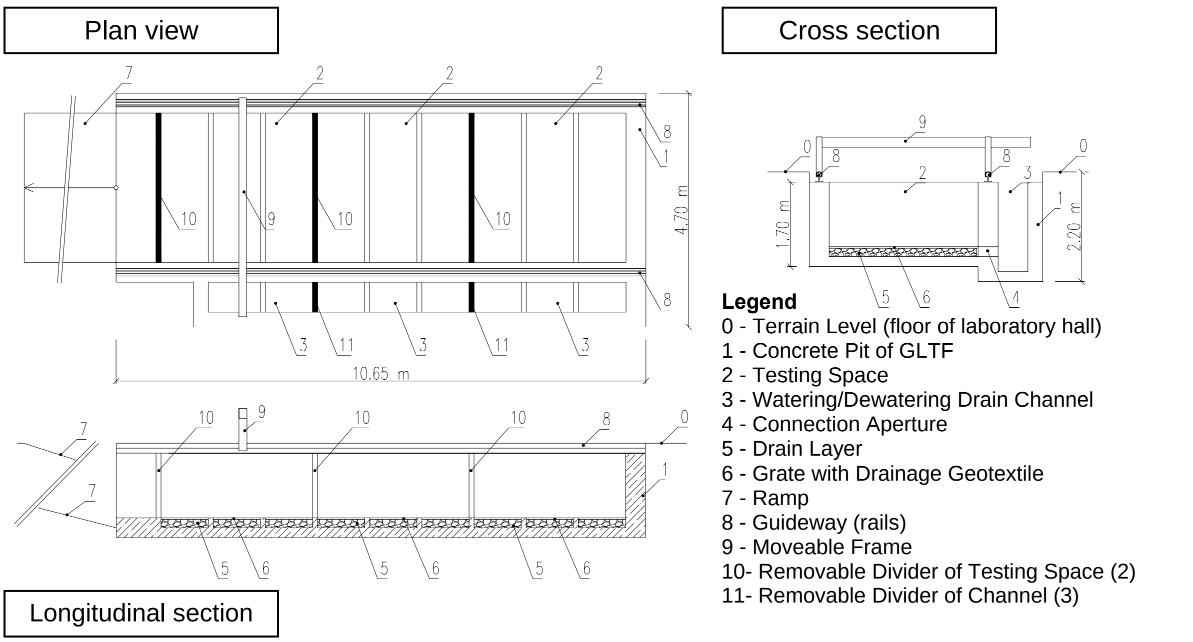

Figure 1: Design of the Geotechnical Laboratory Testing Field (GLTF)

2.1 Tests arrangement

As indicated above, the tests were carried out in the GLTF that had been divided into three testing spaces of a same size (3 m × 3 m). There were five series of tests. One series means two geosynthetics measuring in one step – one by one in each of two testing spaces (laid on subgrade), and in one testing space that was kept without geosynthetics for a comparison. The first and the second series of testing were done on a subgrade with modulus of deformation about E_{v2} = 5 MPa, and the third, fourth and fifth series were carried out on a subgrade with modulus of deformation about E_{v2} = 15 MPa. The subsoil was spread and compacted into GLTF layer by layer up to the final thickness of 70 cm. The material of the subsoil layers was the same for both 5 MPa and 15 MPa subgrade moduli (different bearing capacities were achieved by the moisture content of the subsoil).

The individual series of testing varied in types of geosynthetics used and subbase layers thicknesses laid on geosynthetics (or directly on the subgrade in case of the geosynthetics-free testing space). The first series had the thickness of the first subbase layer 20 cm and the second subbase layer 20 cm (i.e., a 40 cm subbase layer in total). The second series of testing had the thicknesses of the subbase layers 15 cm + 15 cm = 30 cm in total. The third, fourth and fifth series had the same thicknesses of subbase layers 20 cm + 10 cm = 30 cm in total. An example of the test arrangement is shown in Figure 2. Parameters of each of the five series of testing are concentrated in Table 2.

Deformation characteristics as bearing capacity view were obtain from the static plate test according to German Standard DIN 18 134 and were measured on subgrade and on the first and on the second subbase layers.

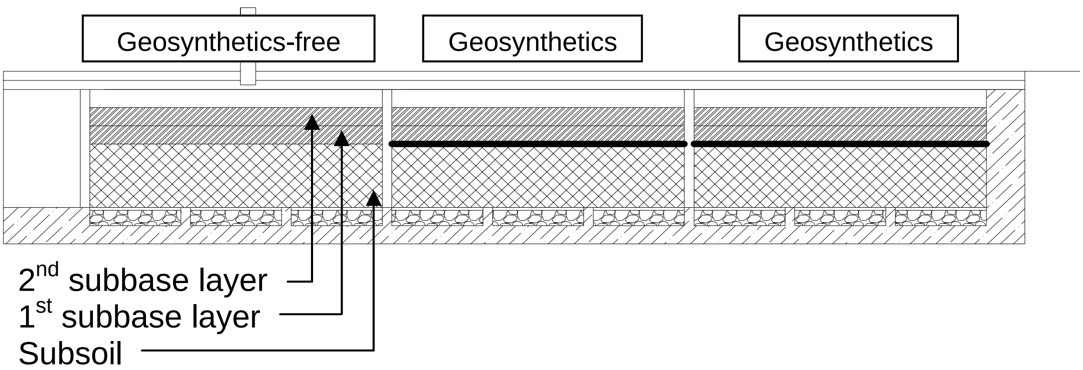

Figure 2: The test arrangement in GLTF

2.2 Material parameters

Used subsoil:

- Weak subsoil was simulated by clayey soil with high plasticity
- Grading:
  - g (2.0 – 60.0) ... 0 %,
  - s (0.063 – 2.0) ... 5 %,
  - f (0.0 – 0.063) ... 95 % = m (0.002 – 0.06) ... 55 % + c (0.0 – 0.06) ... 40 %
- Moisture content: w_{nat} = 28.9 %, $w_{opt,PS}$ = 21.5 %
- Plasticity: w_L = 51 %, w_P = 17 %, I_P = 34 %, I_C = 0.64

Used material for subbase layers:

- Unbound gravel material (fraction 0 – 32 mm)
- Grading:
  - g (2.0 – 60.0) ... 70 %,
  - s (0.063 – 2.0) ... 27 %,
  - f (0.0 – 0.063) ... 3 %
- Particles: d_{10} = 0.3, d_{30} = 2.0, d_{60} = 9.0, C_U = 27, C_C = 1.65
- Density: $\rho_{d,min.}$ = 1.669 kg.m^{-3}, $\rho_{d,max.}$ = 2.208 kg.m^{-3}

Tested geosynthetics are listed in Table 1.

Table 1: Tested geosynthetics

Symbol	Name of geosynthetics	Producer	Type of geosynthetics
A	Armatex G PET (+PVC) 55/55	Kordárna	woven geogrid (flexible)
F	Fornit (PP) 40/40-35 T	Huesker	woven geogrid (flexible)
G	Geolon PP 60	Nicolon	woven geotextile
P	Polyfelt TS 700	Polyfelt	non-woven geotextile mechanically and heat-solidified
S	Secugrid 60/60 (PET)	Naue Fasertechnic	geogrid (welded strips)
T30	Tensar SS30	Tensar International	extruded geogrid (rigid)
T40	Tensar SS40		extruded geogrid (rigid)

Composition of tests through the five testing series is displayed in Table 2.

Table 2: Composition of tests through the five testing series

Testing series	Average subgrade modulus E_{v2} (MPa)	Subbase layers thickness $1^{st} + 2^{nd}$ = total (cm)	Tested geosynthetics(* (see symbol in Table 1)
1^{st}	5	20 + 20 = 40	G + T30
2^{nd}		15 + 15 = 30	S + P
3^{rd}	15	20 + 10 = 30	F + A
4^{th}			G + T30
5^{th}			T40 + S

(* As described in paragraph 2.1 one testing series consists of testing two geosynthetics and a comparison measurement on the geosynthetics-free testing space.

2.3 Results

Table 3: Test results of static tests

Testing series	Subgrade deformation modulus E_{v2} (MPa)	Testing space of GLTF	Symbol of tested geosynthetics (see to Table 1)	Subbase			
				1^{st} layer		$1^{st} + 2^{nd}$ layer	
				thickness (cm)	E_{v2} (MPa)	thickness (cm)	E_{v2} (MPa)
1^{st}	5	I	Geosynthetics-free	20	17.90	40	53.66
		II	G		23.23		56.51
		III	T30		24.90		55.41
2^{nd}	5	I	S	15	11.30	30	22.06
		II	P		9.94		17.43
		III	Geosynthetics-free		10.11		18.18
3^{rd}	15	I	F	20	27.18	30	34.82
		II	A		29.79		37.06
		III	Geosynthetics-free		28.43		38.53
4^{th}	15	I	G	20	25.17	30	33.28
		II	T30		29.79		34.86
		III	Geosynthetics-free		28.43		34.20
5^{th}	15	I	T40	20	25.94	30	35.19
		II	S		27.87		35.54
		III	Geosynthetics-free		29.26		37.74

The pre-described values of subgrade deformation modulus 5 MPa and 15 MPa for the 1st and 2nd testing series and for 3rd, 4th and 5th testing series respectively were achieved with difficulties by water content changing in the subsoil. The modulus was measured three times in each testing space. Values 5 MPa and 15 MPa are rounded off from the average value (e.g., the minimum value was 5.75 MPa and the minimal value was 4.37 MPa in the first series).

The first subbase layer was spread just after subgrade modulus measurement due to the subsoil drying up. Subbase modulus of deformation was measured again three times in each testing space. Values in Table 3 are the average values of the deformation modulus in each testing space.

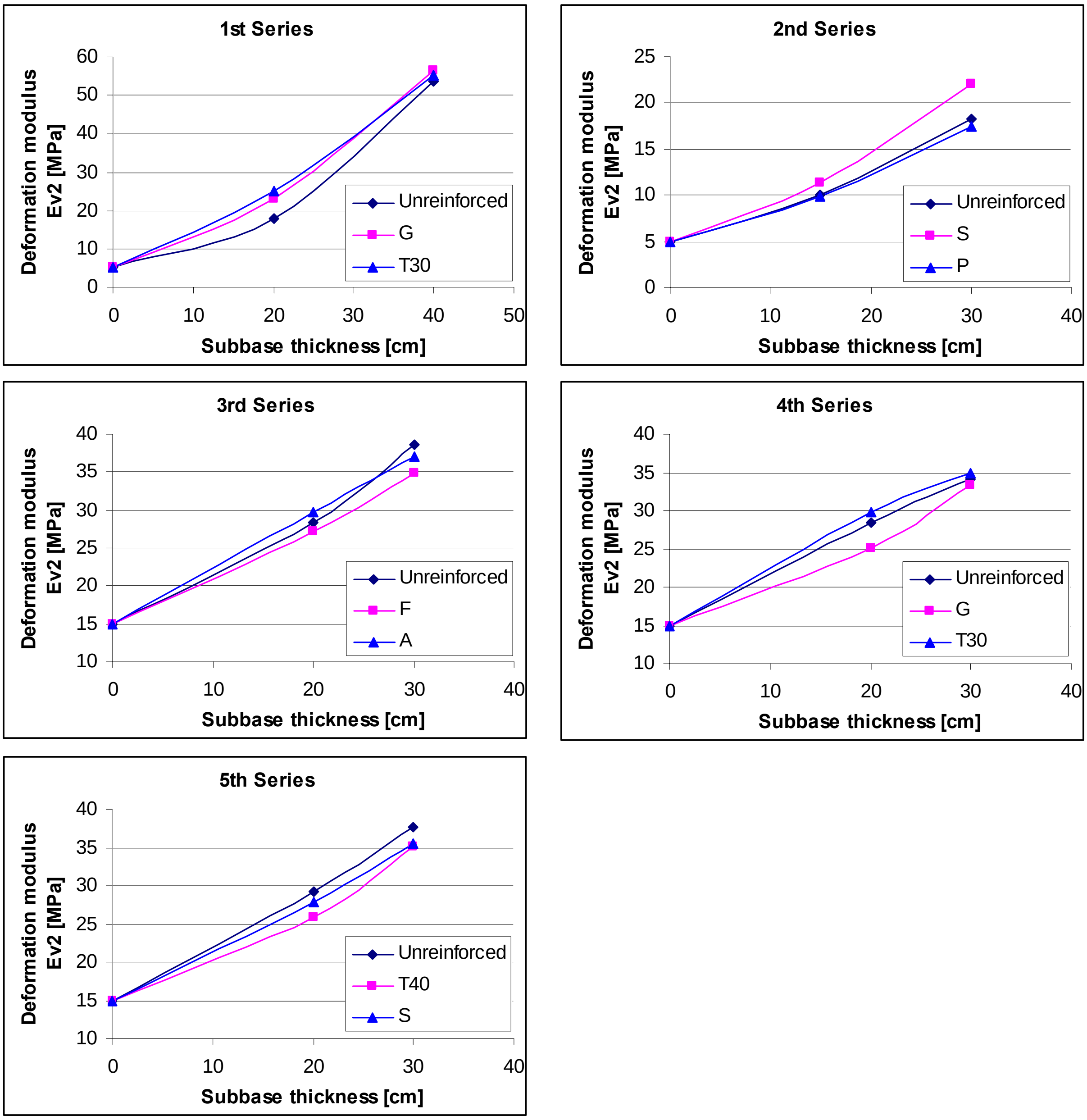

Figure 3: Results of static tests

The test results of the five testing series are concentrated in both Table 3 and Figure 3. At this point it is necessary to highlight again that the thicknesses of the subbase layers vary from series to series. They are 20 cm for both 1st and 2nd subbase layers in case of the 1st testing series, 15 cm in case of 2nd testing series and in case of 3rd, 4th and 5th series the 1st subbase layer was 20 cm and 2nd layer was 10 cm (compare with Table 2).

While the subgrade values 5 MPa and 15 MPa had been chosen as representatives of very weak and weak subgrades before measurement, the subbase thicknesses were adjusted during the measurement to ensure the best geosynthetics effect on the bearing capacity.

2.4 Discussion on static tests results

As demonstrated in Table 3 the bearing capacity increase due to geosynthetics usage is significant on the first 15cm subbase layer spread on geosynthetics laid on the very weak subgrade of 5 MPa deformation modulus only (see 1st series and 1st subbase layer measurement). Partly the same effect is visible in the 2nd series (geosynthetics “S” vs. geosynthetics-free testing space).

The other testing series did not demonstrate any effects of geosynthetics on the bearing capacity increase. The test results of these measurements are the same (geosynthetics used or unused) within the frame of statistical discrepancies.

On the other hand, the explanation of geosynthetics behaviour as a tool of bearing capacity increase is based on the static plate test only. This presumption is fully causative, because the plate static test is widely used for the evaluation of newly-built subgrade in many European countries (Germany, Austria, Czech Republic, Slovakia, etc.). However, this test cannot take into account future construction behaviour, which seems to be very important in the case of geosynthetics, whose incidence could increase through the exploitation time in a construction. Therefore, testing under cyclic loading was arranged.

3 ENVIRONMENT, PARAMETERS AND RESULTS OF TESTS UNDER CYCLIC LOADING

Considering the results of the static tests it was decided to continue with the observation of geosynthetics behaviour under cyclic (dynamic) loading. With respect to the GLTF parameters, cyclic loading via a circular plate with a diameter of 30 cm (the same as for the static plate test) was chosen. For this purpose, it was necessary to design, fabricate and open a new facility – the PneuTester. Analogous facilities were used by Penner, 1985 and Perkins, 2002.

3.1 Facility

The facility for cyclic testing (except the GLTF, see Chapter 2) consists of:

1. Pneumatic loading system (PneuTester, see Figure 4):
   - source of compressed air – compressor Schneider 850-270 ST
   - compressed air set-up unit FRC-1/2-D
   - pneumatic piston chamber DNG 250
   - proportional reducer, pass valve, small items (silencers, connecting hoses, etc.)
2. Computer based electronic measuring and control system with soil and geosynthetics stress gauges, strain indicators, software.

The PneuTester facility is displayed in Figure 4.

Figure 4: The PneuTester in the GLTF

3.2 Tests arrangement

The research plan was composed as described hereinafter and the tests and their methodology are still in progress. The GLTF was divided into three testing spaces of the same size (3 m × 3 m) again. It is supposed that ten series of tests will be carried out on subgrade with deformation modulus E_{v2} = 5 MPa and 15 MPa as within the static tests. Up to now, two testing series have been carried out (the meaning of a testing series is the same as described in Chapter 2).

As in the case of the static tests, the subsoil was spread and compacted into GLTF layer by layer, up to the final thickness of 70 cm. The material of the subsoil layers was the same as described in Chapter 2.2.

The individual series of testing varies in the types of geosynthetics used. The thicknesses of subbase layers are (will be) the same in all series 20 cm – 1^{st} layer and 10 cm – 2^{nd} layer, i.e., 30 cm total thickness of the subbase layer.

Parameters of the cyclic loading was determined with respect to the similarities to real traffic loading: stress under the loading plate – 0.4 MPa, loading frequency – 0.5 Hz, number of cycles – min. 100 000 (informatively in some cases 500 000 and 1 mil.).

3.3 Measured quantities

Within the tests the following quantities were measured:
1. Modulus of deformation E_{v2} – before and after cyclic loading
2. E – modulus – continuously during the tests
3. Contact stress and deformation – continuously during the tests
4. Tensile stress of geosynthetics – continuously during the tests
5. Stress on subgrade level under the plate centre
6. Deformation on subgrade level under the plate edge (2 sensors)
7. Deformation at the top of subbase layer in horizontal distances of 20 cm, 40 cm and 60 cm (from the plate centre)
8. Air temperature and temperature of the sensor support

3.4 Results

The basic outputs are in the form of graphic relationships between the deformation under the plate and the number of loading cycles. A summary of the test results is shown in Figure 6. As the purpose of our paper is not to evaluate the influence of geosynthetics of selected producers, we do not specify which curve in the graph of Figure 6 relates to specific geosynthetics. The graph should be understood as a demonstration of the selected geosynthetics influence on the deformation of the subbase made on soft subgrade. It could also be stated that the geosynthetics-less testing space demonstrated a deformation of up to 3 mm as is visible in the graph. If we take in to account this finding, it is possible to say that the majority of geosynthetics increases deformations of the subbase made on soft-soil.

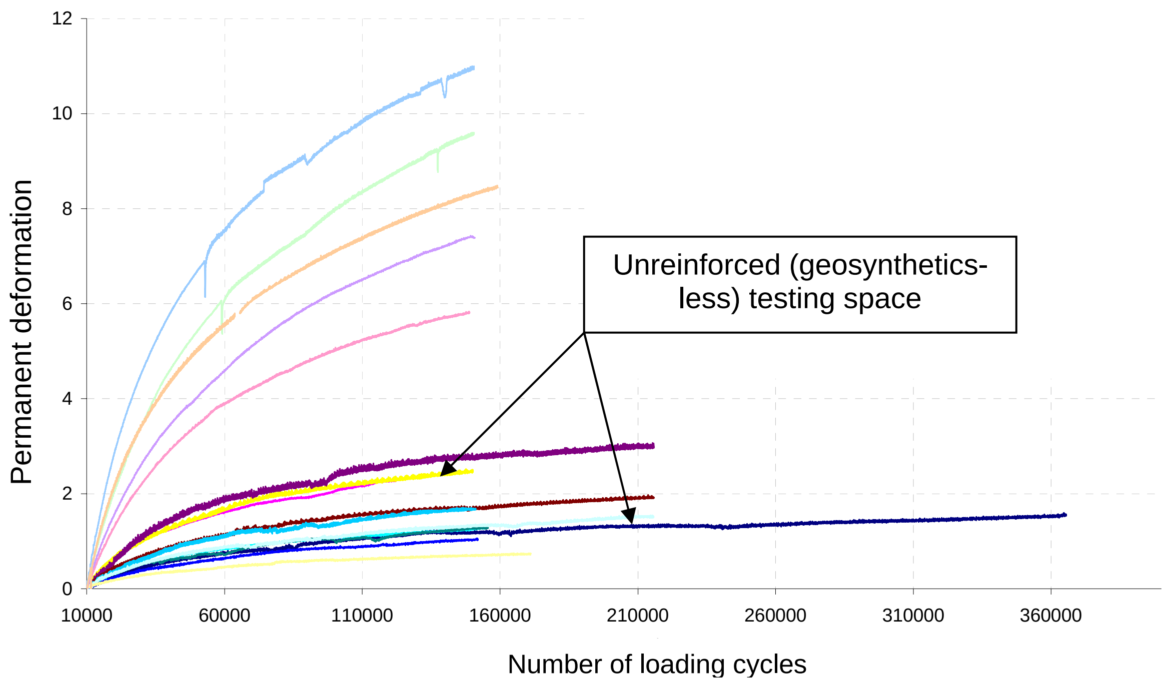

Figure 6: Dependence of subbase deformation reinforced by selected geosynthetics on a number of cycles – unreinforced subbase showed deformation up to 3 mm

The tests summarised in Figure 6 were carried out on the 1st subbase layer (i.e., 20 cm thick – see Figure 2). Deformation moduli of the subbase in all cases were before tests (27 ± 3) MPa and (53 ± 3) MPa after tests (after loading). Tensile stress of geosynthetics caused by cyclic loading was not observed.

3.5 Discussion on tests under cyclic loading

As demonstrated in Figure 6 it was found that a reinforced subbase had, in the majority of cases, a higher deformation within loading than unreinforced subbase. This, perhaps, unexpected result, shown exemplarily in Figure 6, was validated through additional experiments with very similar results.

The two curves in Figure 6 assigned to the unreinforced subbase (representing repetitive two tests) show that geosynthetics-less formation has more or less the same deformation as reinforced formation uses the “best” geosynthetics. Using this finding it is possible to formulate the following: Geosynthetics spread on weak soft soil have an indifferent or worsening influence on subbase deformation and deformation modulus.

4 CONCLUSION

Full-scale tests are preferred for their predicative ability, as they are not affected by model testing inaccuracy. Their results are useful for back-calculation when defining calculation processes and for designing constructions.

The results of the static tests demonstrated the unbeneficial influence of selected geosynthetics on increasing bearing capacity. To this conclusion there were opponent statements expressed during the scientific conferences (e.g., 7th International Conference on Geosynthetics in Nice, 2002, and BCRA conference in Trondheim, 2005).

According to the opponents, the influence of geosynthetics increases with their exploitation. They presume that traffic loading causes toothing aggregate in to geogrids and by this effect geogrids are able to mediate tensile stress. It can be stated that perhaps in the case of some geosynthetics described effect causes a slightly lower deformation of the subbase surface after the loading tests, in comparison with unreinforced soil, even if the tensile stress of the geosynthetics was not detected. We do not understand this lower deformation as an influence of the geosynthetics but a random event caused within the frame of accuracy of the testing method. The influence of the "toothing effect" on the bearing capacity expressed by the deformation modulus measured after the tests has not been observed.

The very speculative benefits in the case of the reinforced subbase pavement layers on weak subgrade, affected by static and cyclic loading tests can be summarized as follows:

- The only significant benefit of geosynthetics usage was demonstrated in the case of the static tests on subgrade with Ev2 = 5 MPa observed on subbase layers of up to 30 cm thick.
- Some of tested geosynthetics shoved out a slight but measurable influence on the final permanent deformation of subbase surface. We do not understand this positive influence as authenticated.

The final conclusion of our findings can be expressed in two topics with respect to the selected geosynthetics, the tests arrangements and real usage (if similar with tests carried out):

- Geosynthetics usage for increasing bearing capacity is not reasonable.
- Geosynthetics usage for deformation decreasing was not authenticated. In the majority of observed cases a worsening influence of geosynthetics was demonstrated. (This conclusion does not concern the usage in the case of unpaved roads where geosynthetics protect from creating deep ruts due to their "membrane effect".
- The behaviour of geosynthetics in observed formations can be understood as a separation layer which causes the sliding of the upper layer on the geosynthetic surface. Subsoil does not work as a formation but as several separated layers which has worse deformation characteristics as a whole collaborating formation.

It should be expressed that our findings do not generally concern all products, as well as other geosynthetics features, i.e., separation, filtration, protecting, slope/embankment-reinforcing, etc., where geosynthetics often play an unsubstitutable role.

The results of the described research are too comprehensive to be published as a journal paper in full text. Therefore the authors are preparing a monograph consisting of all the findings and descriptions.

5 ACKNOWLEDGEMENT

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Pollution of the Highways Runoff

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ABSTRACT: This paper collates the final results of the project dealing with the quantity and quality of highways rainfall – runoff. Field investigation was carried out through the period 2005 – 2007 on several stretches of D1 highway Praha – Brno. Low contents of the EU priority dangerous substances in surface runoff, which depends on the character of sampling and level of traffic intensity, were found. An impact on the water ecosystem, mainly on the algae *Scenedesmus quadricauda,* was confirmed through ecotoxicity testing. The measurement of precipitation and outflow has also brought findings about the variability of runoff coefficient in this built up transport area.

KEY WORDS: Highways, runoff, water quality, PAH, heavy metals, ecotoxicity.

1 INTRODUCTION

In this section the results of the project VaV 1F54G/011/120 “Influence of highways and motorways on rainfall-runoff relations and their impact on water bodies pursuant to Directive 2000/60/EC” are given. The project was supported by Czech Ministry of Transport and dealt with quality and quantity of water flowing off from highways. The basic questions were defined as: What is the impact of highways and motorways on the receiving bodies and water bodies? Which are the main pollutants? In what quantities do the priority dangerous substances specified by EU occur? What are the recommendations and proposals for measures?

2 GENERAL

Pollution of the basic components of the environment (air, soil, water) caused by car traffic is very topical in the CR. The main causes are an ever growing density of traffic in cities and on motorways and highways, but also the further development of transport infrastructure and hundreds of kilometers of new highways which are being built.

Transport emissions with dangerous substances have a negative impact not only on the environment but also on human health. The intensity of pollution is connected mainly with traffic density, amount and composition of fuel and on the type of road. The type and condition of an engine and regime of driving are also important.

Many of emitted pollutants, e.g. polyaromatic hydrocarbons, cadmium, lead, mercury, nickel, are grouped in the catagory of priority dangerous substances specified by EU. The EU member states have a duty to eliminate their occurrence in the water ecosystem.

The main transport processes of this pollution in the environment are atmospheric dispersion; wash out from the road surface and splashing of small dispersion. It concerns mainly metals such as Pb, Cd, Hg, As, Ni and other toxic metals and their compounds, carcinogenic and mutagenic PAH, but also benzene, persistent organic pollutants, dioxins, nitrogen oxide, etc., carbon oxide, particulate matters smaller than 10 or 2,5 micrometers.

The other type of emission is caused by asphalt and tire abrasion, corrosion and leakage of liquids from cars, dumped waste and numerous car accidents also contribute to pollution.

Use of de-icing chemicals in providing safe driving conditions during the winter months causes specific water quality problems, mainly high chloride content and an increasing dissolved (toxic) fraction of metal. During snowmelt and rainfall periods it comes to the outflow of suspended solids with accumulated pollutants. Suspended solids are accumulated gradually in the drains and also in receiving water.

3 METHODS

Monitoring was performed in the period of 2005 – 2007 on the highways D1 Praha-Brno between 61.5 and 81.5 km. Intensity of transport on this stretch is approximately 40 thousand cars/24 hours. Water quality was monitored on the inflow to the storm water sediment basins (SWSB) and in adjacent recipient. Also samples of snow and sludge on the bottom of these basins were analysed. The second monitored profile with the same density of traffic with automatic sampling was situated on the highway bridge on the 149.5 kilometer of D1. The third monitored area was the new stretch at 233.0 kilometer of highway D1 with a very low intensity, and which has been operating only for a short period.

In the samples of water and leach of settled sludge basic chemical parameters, priority substances, etc., were analysed, and also ecotoxicity testing was carried out. The water samples were collected and analyzed according to the standard operation procedures for the sampling and analysis of water - Technical standard of water management and ISO/EN standards.

Figure 1: Profile on the storm water sediment basins (SWSB) on 61.5 km of D1

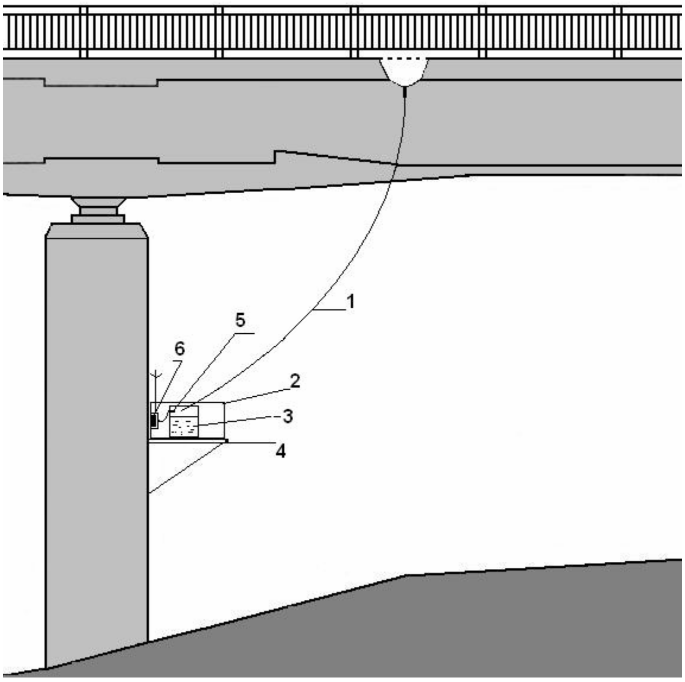

Figure 2: Localization and scheme of an automatic sampling device

Explanation: 1 tube, 2 plastic container, 3 sampling vessel, 4 consoles 5 water level contact maker, 6 transmitting program unit

4 RESULTS OF WATER QUALITY

In Table 1 the results of realized measurement (stretch 61.5 – 81.5 km of D1) and a comparison with the limits of Regulation of. Gov. 229/2007 and Working qualitative limits (2005) are presented. The limits were used in the Czech Republic for period of characterization of water bodies. This process is a part of the implementation of Regulation 60/2000 EC (Water framework Directive – WFD) and preparation of River Basin Plans.

Table 1: Parameters of water quality of highways runoff

Parameter	Unit	Average	Median	Q90	Reg. 229/ 2007	Qualitative limits (2005)
Pb	$\mu g.l^{-1}$	**3,82**	2,40	**6,10**	14,4	5
Cd	$\mu g.l^{-1}$	**0,406**	0,190	**0,770**	0,7	0,2
Ni	$\mu g.l^{-1}$	**45,3**	21,8	**132**	40	5
Hg	$\mu g.l^{-1}$	**0,199**	0,140	**0,270**	0,1	0,1
Cr	$\mu g.l^{-1}$	**4,83**	4,50	**6,80**	35	2
Cu	$\mu g.l^{-1}$	**19,0**	13,7	**52,8**	25	2
Zn	$\mu g.l^{-1}$	**142**	69,0	**400**	160	10
Cl	$mg.l^{-1}$	**1095**	726	**1 510**	250	-
Hydrocarbons C10-C40	$mg.l^{-1}$	**0,145**	0,145	**0,88**	0,1	-
Benzo(b)fluoranthene	$ng.l^{-1}$	**7,66**	3,75	**20,4**	60	30
Benzo(k)fluoranthene	$ng.l^{-1}$	**5,87**	3,65	**15,7**	60	30
Benzo(a)pyrene	$ng.l^{-1}$	**5,63**	2,10	**11,8**	100	50
Benzo(g,h,i)perylene	$ng.l^{-1}$	**6,29**	3,33	**13,1**	30	16
Indeno(1,2,3-cd)pyrene	$ng.l^{-1}$	**5,69**	3,25	**15,5**	30	16
Fluoranthene	$ng.l^{-1}$	**21,2**	9,80	**63,0**	200	90
Σ 6 PAH	$ng.l^{-1}$	**7,66**	3,75	**20,4**	200	-

Q90 – value of 90 % of exceeding

It is possible to say that the highest concentrations in runoff were found in the samples from the automatic sampler. Other waters probably dilute other samples of water from the rainfall edimentation basins.

Almost high-risk pollution levels were found in the samples of settled sludge as a consequence of accumulation and fixation of organic compounds and metals to particulates. In Figure 3 the results of the concentrations of phenanthrene, pyrene, fluoranthene and fluorene can be seen from all monitored profiles including sampler.

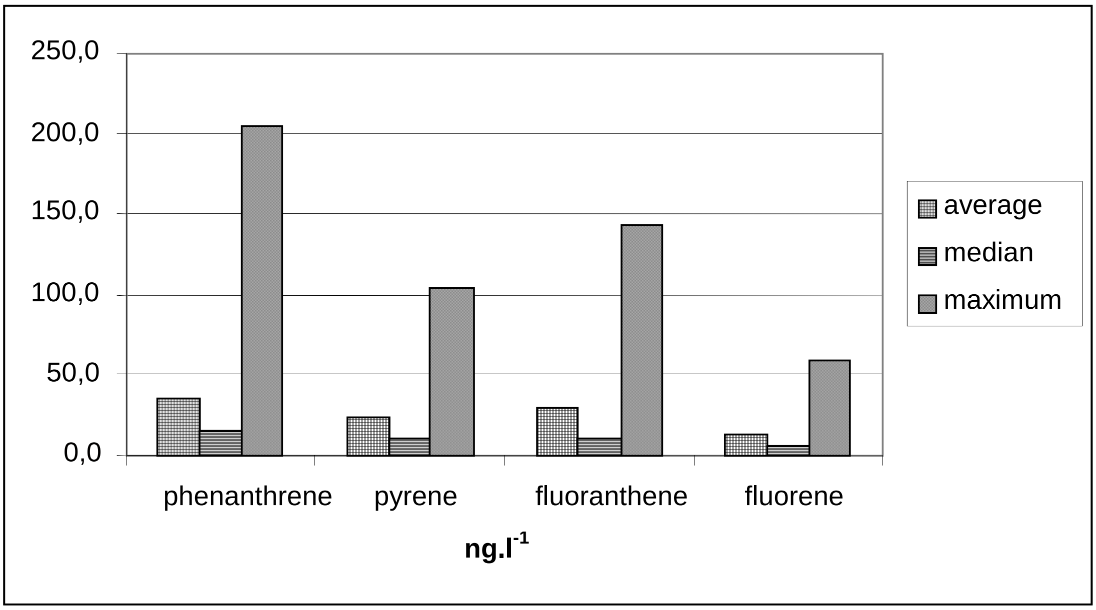

Figure 3: Results of contents of PAHs in highways runoff

Results of ecotoxicity testing has confirmed that acute toxicity is positive, mainly by testing algae *Scenedesmus quadricauda,* in comparison with the testing on invertebrate *Daphnia magna*, and is connected also with higher concentration of chloride in water.

5 RAINFALL - RUNOFF CONDITION

The measurement and evaluation of rainfall-runoff condition was carried out on the object storm water sediment basins (SWSB) on 72.1 km of highway D1 Praha-Brno with the 5,375 ha of dewatering area. The course of rainfall events was registered by a self-recording gauge. Inflow and outflow from basin were measured by Poncelet and Parshal devices. Data were registered and communicated. In the next picture the rainfall events and also the response in the catchments area are shown.

For rainfall cases the average outflow coefficients were (0.53 – 0.87), showing a share between outflow volume and rainfall derived. Consequently a specific surface runoff was calculated, on the unit of highways dewatering area. The value was used for the mathematical modelling of load and concentration of chloride in two recipients.

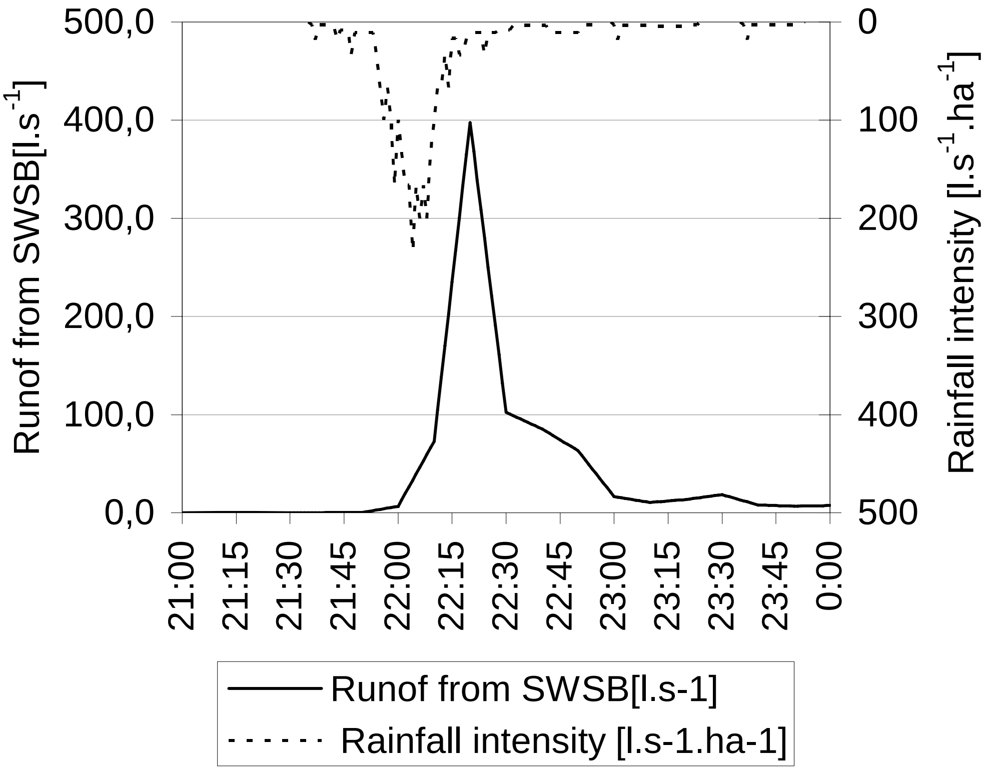

Figure 4: Response in the catchments - rainfall - outflow 12. 9. 2005

6 POTENTIALLY THREATENED WATER BODIES

One of the outcomes of this project was the selection and assessment of potentially threatened water bodies by run-off waters from the highways in Czech Republic. For this model, analysis tools of spatial analysis of Geographic information systems (GIS) and multi-criterion assessment were used. Selected factors entering the analysis are represented by polygons (climatic conditions – long term precipitation amount, altitude, soil conditions, protective zone of water resources, protected area of natural water accumulations), poly-lines (highway trace through water body, traffic intensity) or points (crossing of water body with highway). The result of this analysis is shown in Figure 5.

This method of assessment is important for the first identification of the areas by using a similar criterion and a specific simplification for them. The recommendations and proposals of protective measures will be elaborated for selected areas with high potential threat.

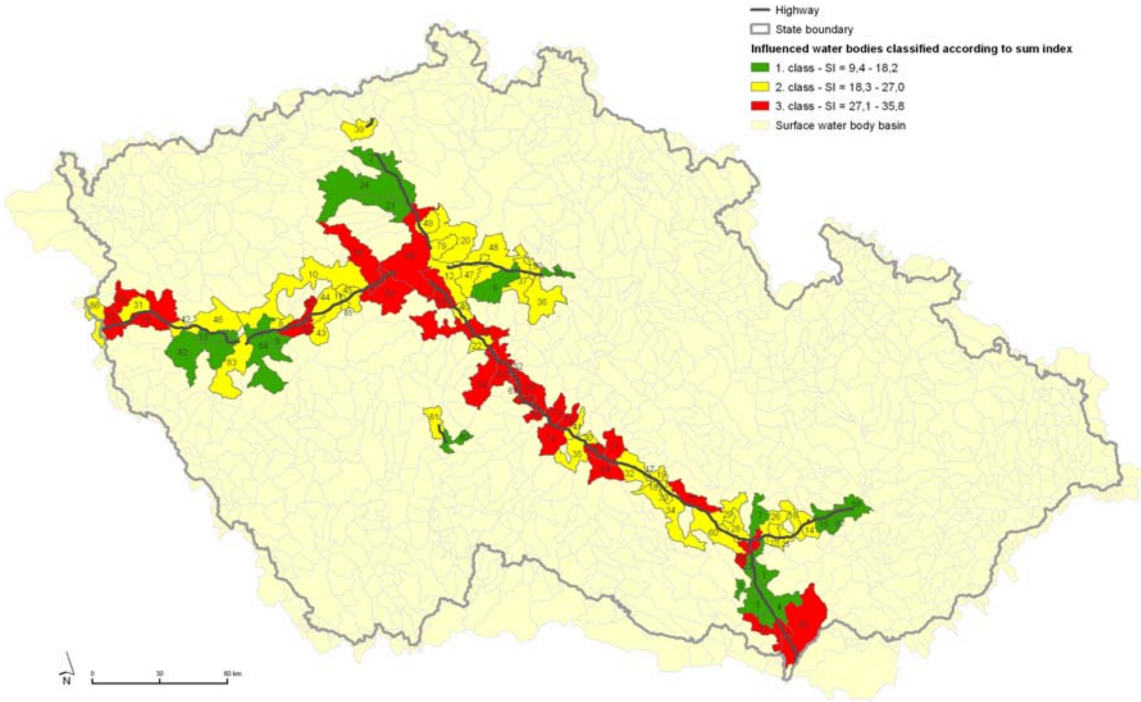

Figure 5: Potentially threatened surface water body basins

7 MEASURES AGAINST CONTAMINATION

Implementing some measure against the discussed contamination is known abroad: Best Management Practices (carrying out periodic maintenance, determination of limits and system of control - BMP). In Germany and other European states there are various systems of secondary treatment of water with infiltration of the out flowing rainfall runoff. The newest type is 3-stages equipment with a soil filter and consequential soil infiltration or draining away into receiving water.

8 CONCLUSION

The findings from this project can be summarized as follows:

The negative impact of runoff from highways on the recipients and water bodies also enhances a certain amount of priority dangerous substance as specified by EU.

High concentrations of chloride from winter road maintenance increase ecotoxicity of water, which was demonstrated by testing on algae.

During intensive rainfall small receiving bodies flood; polluted suspended solids are diluted and transported into the river basin.

Similarly, as in other European states, it seems to be necessary to monitor and control this potential strain of pollution along the highways and to do the BMPs and to project and implement protective measures against it.

9 ACKNOWLEDGEMENT

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Key Attributes of the High Speed Rail System Project

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ABSTRACT: The contribution focuses on the provision of the high speed railway lines (HSRL) in a region, in which the HSRL is being considered. Defined first is the high speed rail system concept, followed by a description of the procedure preceding the implementation of such a railway line, and the basic principles of the HSRL route layout.

KEY WORDS: High speed railway line, rail route layout.

1 HIGH SPEED RAILWAY LINE CHARACTERISTICS

Since this paper deals with the high speed railway lines (HSRL), it would be appropriate to first describe such definitions as accurately as possible, as well as to explain in which aspects the HSRL differs from other railway lines.

Rail systems can be grouped by their different features. In terms of physical features, two differing features can be mentioned that distinguish the two given systems that enable movement of a railway carriage on a long distance railway line – these are the adhesion railway and the railway based on the principle of the magnetic levitation, generally referred to as maglev.

For proper functioning of the adhesion railway, contact between the wheel and the track is necessary. In the case of the maglev, the carriage moves free from contact above its roadway as the result of the magnetic levitation (based on application of the linear electromotor). In commercial services, the maglev operates only in the city and suburban transportation, but projects to also construct long distance railway lines have already been prepared. The advantages of the maglev are the high operating speed, fast acceleration and deceleration (which shortens the travelling time), non-existent mechanical wear of the rails during the carriages' movement, and noise, which is only created by the aerodynamics; its disadvantages are the incompatibility with other transportation systems (creating difficulties in building such rails in stages, as well as cooperation with other types of transport means) and impossibility of the track construction on the earthwork structure.

In terms of the highest trains' speed facilitated by the railway, it appears that the most suitable grouping is the one in terms of the two Council Directives on Interoperability of the Transeuropean Railway Systems (Council Directive no. 96/48/EC, 1996; Council Directive no. 2001/16/EC, 2001), which set out differences between the conventional and the HSRL. The HSRL includes specially constructed HSRL for speeds up to 250 km/h and higher, specially modernised railway lines for speeds of the magnitude of 200 km/h, and specially modernised rails of unusual characteristics given by topographic, terrain or urban limitations, to which the speed must be adjusted in each individual case separately (Council Directive no. 96/48/EC, 1996).

Considered as a new, high speed railway track is therefore such adhesion standard-gauge railway (track gauge 1 435 mm is the basic condition of a railway carriage's smooth crossing between various railway lines) which is chiefly intended for the long distance transport (the railway line length is in the order of hundreds of kilometres), and of which line speed is at least 250 km/h; together with the modernised sections usually rated up to the speed of 200 km/h; these rails then create a high speed railway network.

2 THE HIGH SPEED RAILWAY LINES BASIC PARAMETERS

To determine the basic parameters of the HSRL, it is first necessary to establish the reason for the HSRL's construction and after that to determine which trains will be using the HSRL. Generally, it is possible to find at least a few closely correlating arguments to support the construction of an HSRL (that is why several justifications usually come up almost simultaneously), and of which the following could be applicable examples (Týfa, 2006):

- insufficient capacity of the conventional railway network (especially within large conglomerations with heavy suburban passenger transport)
- slow train speed (long travelling times) by the conventional network (the rail transport is not attractive either to the passenger, or to the transporters, i.e., it is not competitive in comparison with road and air transport)
- heavy transport streams (having potential for further growth) of some connections, which are presently carried out using different transportation means
- unreliability of the conventional trains (failure to adhere to the train schedule due to breakdowns and extraordinary situations), and provision of only low comfort levels to passengers (again, lack of the rail transport's attractivity and loss of customers)
- independence of non-renewable energy sources – crude oil (this reason will become increasingly more current in the forthcoming decades; the crude oil crises in the 2nd half of 20th century contributed towards the development of HSRL)

One or more of the following train types fall under consideration to be used as the HSRL service (Týfa, 2006):

- special high-speed passenger train units, providing long distance transport at designed speeds of (300 - 350) km/h
- long distance (alt. regional) passenger trains, consisting of a locomotive and passenger carriages (plus optional controlling carriage), which are capable of reaching speeds of (160 - 200) km/h
- Transeuropean passenger express trains, transporting at the same time the travellers' passenger cars (so called trailer-trains) consisting of a locomotive, sleeping and couchette carriages and special freight wagons to transport passenger cars, and which are capable of reaching speeds of (120 - 160) km/h
- special freight train units to transport post (and alternatively other similar consignments) of designed speeds of (300 - 350) km/h
- freight trains intended especially for the provision of unaccompanied combined freight transport, which are capable reaching speeds of (120 - 160) km/h

The selection of the train types and their parameters plays a key role in terms of the evaluation of the investment to be expended, including regular operating costs required for the maintenance and operation of the railway line, when usage of HSRL is to be considered – its attractivity to the transporters.

To be able to determine the basic design parameters of the HSRL, it is necessary to know at least the characteristics of the afore-mentioned individual train types (and not only of their driving units):

- max. speed, which the train is able to achieve
- max. acceleration to reach the train's max. speed and the braking distance from the top speed
- max. longitudinal gradient of the railway line, on which the train is able to maintain its highest speed, alternatively max. speed, which the given train is able to maintain on such longitudinal gradient, and which is the highest applicable to the highest performance train assumed
- grouping of the train to the loading class (max. mass on the axis and the unit of the carriage length)
- max. length of the train

To determine the design parameters of the railway line it is not crucial as to how the specific category of the train will be marked or what the exact route of the railway line will be, but which characteristics will correspond to the given group of trains. On the basis of the knowledge of the described train types' characteristics and in conformity with the technical norms and legal regulations, it is possible to determine the critical parameters of the HSRL layout as follows:

- min. radius of the horizontal curve
- max. longitudinal gradient of the railway line
- min. effective length of the running track in the HSRL operating control points
- min. length of the platform edge of the HSRL stations' platforms

A necessary condition to be considered applicable to carriages of all train types is their assignation to rails with normal gauge, and the adherence to the limited dimensions of the carriage's contour in accordance with international standards. Equipment of the railway line comprising solid parts of the interlocking system and the electric traction can be in principle adapted to any HSRL route. All HSRL parameters must be in conformance with the Technical Specification of Interoperability (TSI) applicable to the Transeuropean high speed rail system.

Upon expert selection of the trains and determination of their parameters it is possible to approach the design of the HSRL routes' options. Once their layout is completed, it is necessary to perform the travel simulation of all train types, in terms of the dynamics of their travel (affected especially by the longitudinal gradient of the individual track sections), and it is through this that the calculation of the travelling time of the train and the traction energy consumption will be carried out, as well as verification that the line speed has been reached.

The bigger the trains' variety and the wider their parameters' range, the stricter the HSRL design criteria becomes; consequently, the search for the optimal route becomes more demanding and its construction more expensive. Big differences between the maximum speeds of the fastest and the slowest trains will manifest itself by the necessity of a large radius of the horizontal curves and increased wear of the railway's superstructure. The traction characteristics of the train, considerably influenced by its mass and the power output of the drive-axle assemblies, will manifest itself directly in the maximum longitudinal gradient of the railway line.

For HSRL combined operation (passenger and freight trains) it is possible to determine the radius of the horizontal curves as approximately 7 000 m, and for the HSRL operation of only special high speed units, a radius of the horizontal curve of approximately 4 000 m will suffice. The smallest admissible values of the horizontal curves' radius are further lowered when a solid track-bed is used in the track construction. The biggest HSRL longitudinal gradient with combined operation can be determined as 18 ‰. An example of the HSRL intended only for the special high speed units are the rails in France, where the TGV units overcome ascend of values of up to 35 ‰, or in Germany, between Cologne and Frankfurt, where the ICE 3 units manage the longitudinal gradient of up to 40 ‰. (Lichtberger, 2005)

Especially within the territory of the Czech Republic, characterised by the complex configuration of the terrain, scattered settlements and a unique natural and cultural heritage, even small changes in the limiting values of the HSRL routing parameters play a key role in the capital intensity of its construction.

3 HIGH SPEED RAILWAY LINES ROUTING

In accordance with the reasons that lead to the proposal of the new HSRL and the train types, of which operation is assumed on the HSRL, the HSRL route connects important residential and industrial conglomerations as the sources and target journeys of the travellers (alternatively goods), replaces sections of the low line speed conventional rails, or increases the almost used up capacity of the existing rails. Routing of the new HSRL is constrained by the limiting design parameters and effort to minimise the investment costs of the construction and the future operating costs, as well as efforts to make the route as short as possible. At the same time the HSRL routing is limited by the availability of the free space between the residential formations, industrial zones and transport constructions as well as the necessity to protect the cultural and, especially, the natural assets of the territory.

To the limiting conditions indicated in the preceding clause have to be added the problems with the location of the passengers' boarding / exit / transfer platforms and the crossing of the trains from the high speed rail network to the conventional one, and vice versa. One of the biggest advantages of passenger rail transport in comparison with air transport is the fact that the train can bring the traveller directly into the city centre, where there is a natural concentration of all services, availability of transfer to interconnecting public transport systems, and about equal accessibility to any place in the city.

One of the options of the HSRL route and a city's connection is therefore termination of the HSRL at the periphery of the residential agglomeration into the conventional network (see Figure 1b), which will facilitate use of the existing railway to enable trains to consequently travel to the central station. However, this solution has two main drawbacks, which must be examined for each specific case. Firstly, it involves extending the trains' travelling time during the travel within the urban area on the existing rails (although reconstructed within available means), especially for the travellers, who are passing through the given city, and secondly, the complications with the saturated capacity of the existing rails, which occurs especially due to the concentration of urban passenger transportation. As advantageous (but, at the same time, costly) a solution appears in the construction of a new railway line, segregated from the other transport systems, through the city centre (i.e., at a different height level – above ground, or more often underground), building a station or a stopping place as close as possible to the city centre, or an important changing transport terminal. This option became practicable, for instance, in Antwerp, Belgium or in Berlin, the capital city of Germany.

Another possibility as to how to provide a link between the HSRL and the residential area is to build a HSRL bypass around the agglomeration (plus connecting, as and when possible at suitable places, the HSRL with the conventional rail network), and build on it a completely new railway station (see Figure 1a, 1c, 1d), which would be a part of the transport terminal, connected to a good quality network of other types of the public transport services, and provided with ample parking space of the P&R type. The advantage of this option, in spite of the longer HSRL route when compared to the preceding option (bypassing the city instead of going through it) is usually a shorter travelling time, as the result of better design parameters of the railway line (higher line speed), lower investment costs (lesser share of tunnels and bridging structures can be expected on the border of the town residential zone and the rural area, and their lower capital intensity), and the possibility of cooperation with the Individual Automobile Transport. For instance this route was taken by France in implementing the Paris east bypass (LGV-Interconnection), bypassing Lyon with the Lyon-St Exupery (TGV) station, or the new station Avignon TGV.

The attractiveness of high speed rail system can also be increased by interconnecting stations built on HSRL with airport terminals (e.g. Paris – Charles de Gaulle, Frankfurt a. M., Shiphol near Amsterdam). The advantage of such interconnection lies in the fact that on intercontinental flights and Transeuropean routes the passengers are carried by airplanes and then, after changing the airplanes for HSRL, they can be easily carried to the centres of the European metropolis with great comfort and in high speed. Regardless of the chosen means of transport, the journey is realized on the basis of a single ticket issued by the transporters involved in the cooperating system; the application of the airplane ticket on the train is considered as “flight at zero level”. Another advantage rests in the lowering of air space loading. For instance, the building of HSRL on the Paris – Lyon route originally posed a threat to air traffic in terms of competitiveness, but later on resulted in mutually satisfactory co-operation.

During the HSRL routing it is necessary to approach its interconnection with other conventional rails with special consideration. These connections, ensuring trains’ smooth crossing between both rail networks, are, on one hand, advantageous for both rail networks, but, on the other hand, can be a source of complications. The contribution brought about by the connection of both rail networks rests in the fact that the travelling speeds of the conventional trains, which use the HSRL for part of their travel, increases, while at the same time the usage of the HSRL capacity increases as well. However, if the rail network interconnections are made in unsuitable places, they can become a potential source of unreliable operation; namely, serious problems can occur in cases when the train, which is supposed to depart to the HSRL at certain exact time, can not do so due to the occurrence of delays in the conventional network. Travel of various types of trains onto the HSRL, which in addition are also leaving and coming onto the HSRL at different places, poses high demands on the processing of the Train Traffic Timetable (TTT), and that is why there is usually not a big enough margin to shift the train route during the dispatch control of the operation. The TTT design simplifies when certain types of trains are routed only during a certain part of the day (e.g., passenger trains mostly during the daytime, freight trains at night time). (Týfa & Vachtl, 2005)

The link between the TTT and the HSRL route (TTT travelling time and track capacity requirements) causes an actual problem in creating efforts for the HSRL parameters to conform rigorously to so-called system travelling times of the Integral Tact Traffic Timetable in the long distance rail transport. This requirement, in some connections, leads to the belief that in these sections it is not the aim to achieve the technically lowest travelling times of the high speed trains, and so it may at first glance appear that all that is required to be connected into the HSRL network is to modernise the existing railway line. However, such cases can be affected by the following pitfalls:

- HSRL are rails primarily intended for high speed trains, which create their own European link system, and therefore they are the changing links between the trains of this type that must be primarily monitored. Creation of the individual high speed lines’ tact is naturally desirable, as it leads, in terms of the passengers, to an increasing attractiveness.
- The high speed trains are supposed to compete with road vehicles travelling on highways, and also with airtraffic – both groups of transport means are trying to shorten their driving and travelling times as much as possible.

- Construction of any new transport infrastructure is a matter of several years, built at high financial costs, creating a perceivable intervention in the landscape, and its assumed lifespan is at least 100 years. This is why it is necessary to always create certain reserves in the newly built rails' parameters, with the foresight of looking to the future, as the carriage stock is developing faster than the construction of the rails and the organisation of the rail transport operation might change even more dynamically.

At the end of this chapter a general procedure of the HSRL design is given: After the technical design of the railway line, determination of the travelling times of all train types and the design of a few versions of the TTT is necessary, including the prognosis of the transport streams. Through such a prognosis an adequate utilisation of the railway line's capacity can be ascertained, followed by the financial evaluation of the whole construction. The financial evaluation of the construction should also include the all-society benefits and negative aspects, i.e., especially the improvement of the regional transport improvements, increased transport safety, increased transport independence from crude oil, removal of some of the transport streams from other transport systems which create an environmental burden; noise and vibration emissions, landscape deterioration (aesthetic, area fragmentation).

4 CONCLUSION

With the development of technology, commerce and tourism in the 2nd half of the 20th century, most countries of the world have experienced a fast growing demand for transport and the increasing demands of travellers and transporters for transport reliability. The automobile and air transport operators have adapted to these requirements in a versatile manner. Rail transport has also had to start offering their customers higher travelling speed, reliability, sufficient range of connections, comfort and complex range of services. During this revival of rail transport it was consequently recognised, among other things, that certain track sections' capacities were not adequate, and also discovered that their routing and technical parameters were inadequate as well. This gradually led to the radical modernisation of important rail routes in many countries around the world and development of a new HSRL.

Construction of the high speed rail system has also seriously been considered a few times in the Czech Republic (and in former Czechoslovakia). So far, preference has been given to modernisation and optimization of the existing rails, which is certainly needed and through which the previously neglected maintenance has been caught up with; the moral lagging of the railway line infrastructure behind the technical progress of its times and the customers' requirements also have to be attended to. But the modernisation as presently perceived can not, in the long term and on a bigger scale, satisfy the needs of the inhabitants and visitors of the Czech Republic in a Europe without national borders. Neither it is able to compete with other types of transport, which are much more harmful to the environment. It also must be mentioned that the modernisation of the conventional rails is not in contradiction with the construction of the new HSRL, but rather to the contrary – they suitably complement each other. The future rests in two railway networks, each with different functions, but closely and mutually cooperating.

The Czech Republic occupies a strategic position in the centre of Europe, which also predestined it to be at the centre of big events and a crossroad of important routes. But if it does not react quickly to the changes taking place in the rail transport in the neighbouring countries (especially in Germany and Austria), which are, among other things, substantially increasing the qualitative and capacitive level of their railway line infrastructure, the natural potential of the advantageous position will remain unused, and this will reflect in the declining level of the whole economy.

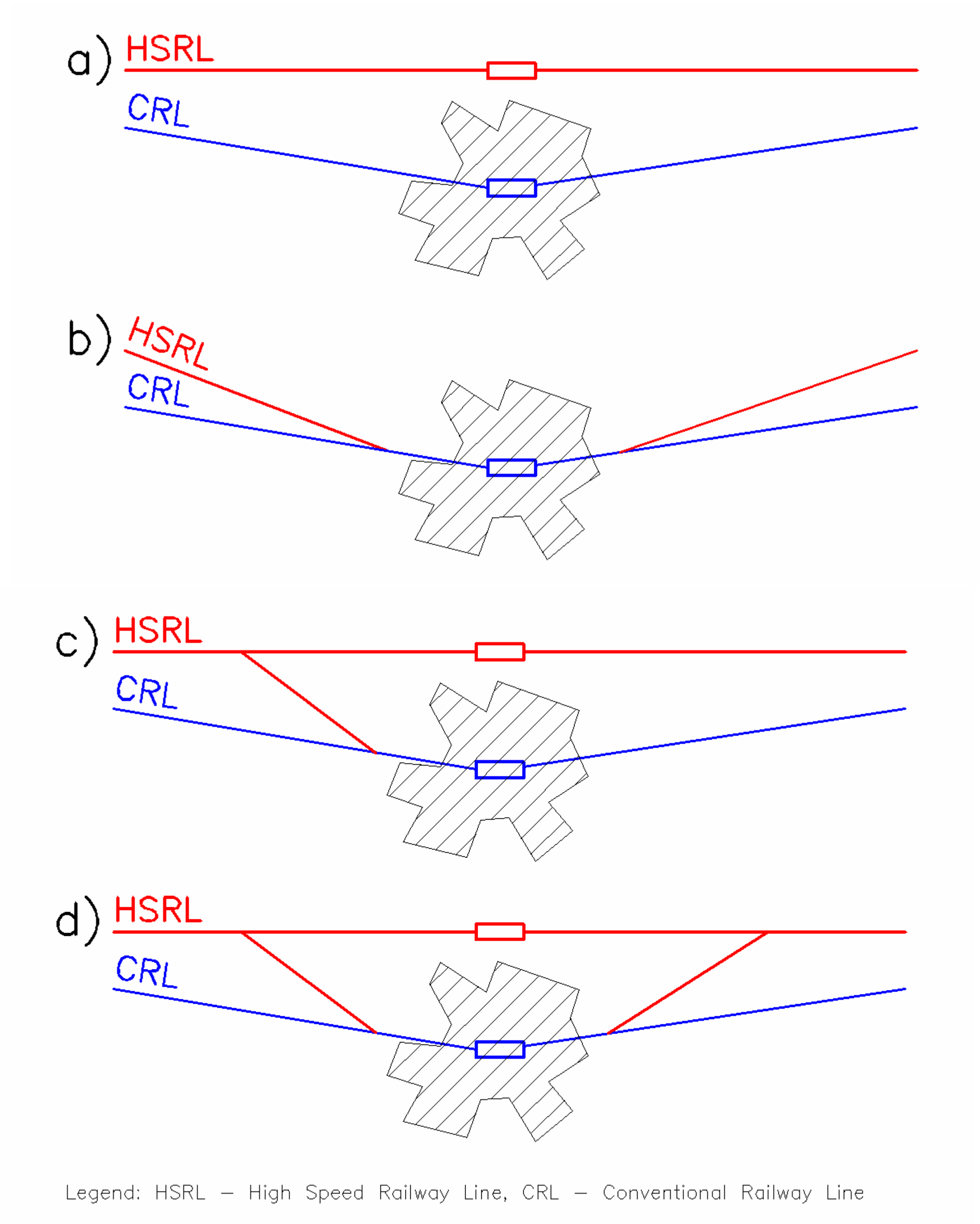

Figure 1: Relation between city and HSRL:
a) HSRL bypass around the agglomeration with a new railway station on it;
b) Termination of the HSRL at the periphery of the residential agglomeration into the conventional rail network;
c) HSRL bypass around the agglomeration with a new railway station on it, plus connecting the HSRL with the conventional rail network on one side of the city;
d) HSRL bypass around the agglomeration with a new railway station on it, plus connecting the HSRL with the conventional rail network on both sides of the city.

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Technical Notes on The I.T. Solution Supports The Independent Railway Infrastructure Manager

1 INTRODUCTION

As a sustainable surface transport, the railway transport is now re-gaining its former position on the transport market. The key "buzzwords" – and, at the same time, the key challenges – are competitiveness, open access for the railway operators, and interoperability. No modern railway organisation can rely upon legacy "PPP" technologies anymore – Phone, Pencil, Paper, mostly due to their disadvantages:

- time-consuming, manual input into paper documents, cumbersome "transmission" and processing
- error-prone when processing and transmitting (distortion during voice communication)
- a high risk of introducing wrong data (e.g., a incorrect engine or train number)
- high latency in data communication (dependent on the instant personal presence)
- cumbersome or impossible further processing (e.g. in GVD Analysis or archiving)

An answer to these challenges of legacy technologies is to replace them with a comprehensive information system based on a digital model of the railway traffic. This model is built upon basic objects, such as a train, an engine, and a closure. The digital model and digital technologies rely heavily on the following conditions, however:

- direct integration into the railway technologies – the IS have to be integrated into existing workplaces and the existing dispatching network, not into a mirrored structure
- network-wide data capture – the information systems should be deployed in the network scope, preferably without any "deaf spots" with undesirable inconsistencies
- full-scope planning of the train traffic (as ensured by the below-mentioned systems)
- coupling with DOZ interlocking systems – when possible, such an integrated solution provides the highest consistency, accuracy, and timeliness of the data

2 OVERALL STRUCTURE OF IS FOR INFRA MANAGER

As in any other enterprise, the infrastructure manager can see his business activities separated into three broad levels, and their support includes the key IT solutions provided by OLTIS Group a.s., as detailed below:

- strategic management – long-term general planning, the infrastructure development (IS of infrastructure), planning the maintenance works and temporary closures (IS Closures), planning the capacity (ISOR KADR), archives and decision support systems (ISOR APD)
- tactical management – mid-term planning, specifying the closure plans (IS Closures), designing timetables (IS KANGO), selling the capacity (ISOR KADR), managing personnel (EDO)
- operational management – short-term, instant traffic control with local scope (IS Station Master), regional scope (ISOR VD), and network scope (ISOR CDS)

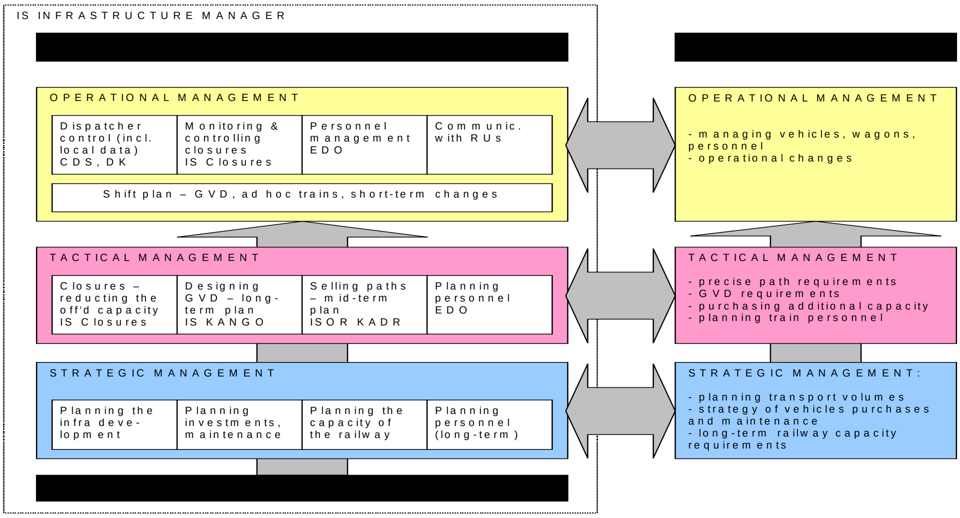

Figure 1: Architecture of the integrated IT solution for infrastructure manager

The overall schema (Figure 1) suggests also communicating with a railway undertaking (RU), on both the process and IT level. Here, any licensed RU can be assumed (passenger or cargo).

The information systems of the operational management can be seen as the mission-critical systems for controlling the train traffic (ISOR CDS – an umbrella system for supervising the whole network, ISOR RVD – distributing the shift plan, Station Master for the local level); however they cannot get along without the support of the other underlying systems concerning mainly data on the network, timetable (GVD), and overall decision support and capacity planning (IS KANGO – timetables, IS Closures, ISOR KADR – selling the capacity).

3 CASE STUDIES: SUCCESSFUL IMPLEMENTATION IN CD AND ZSR

Czech and Slovak Railways (CD and ZSR) have grown from the same environment (CSD), which includes identical or similar vehicles, operational technologies, regulations, and others. The similar environment simplifies the implementation of the IS. In both railway networks, the ISOR family is deployed with a Station Master (DK) at its local level. The systems rely heavily on data communication among their various components or with external IS. The local level communicates with the interlocking systems (SZZ-ETB, ESA, and DOZ).

The OLTIS Group company, based in Olomouc, Czech Republic, is a leading vendor of the IS for controlling the railway transport, from preparing and planning the transport to real-time monitoring and assessment. The company is also a leading exporter in the railway IT industry.

For more info: official web pages of OLTIS Group a.s., http://www.oltis.cz/index.m?lang=EN
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Instructions to the autors

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Papers based on accepted abstracts and prepared in accordance to these guidelines are to be submitted through the journal’s web site www.transportsciences.org.

All papers, using Microsoft Word2000 (or newer) are **limited to a size of at least 4 and no more than 8 single-spaced pages** on A4 paper size (297 mm X 210 mm), including figures, tables, and references and should have an **even number of pages**. The paper’s top, bottom, right and left margins must be 2.5 cm. No headers, footers and page numbers should be inserted.

2 TITLE, AUTHORS, AFFILIATIONS

The title of the paper must be in title letters, Times New Roman, font size 16, and aligned left. Use more than one line if necessary, but always use single-line spacing (without blank lines).

Then, after one blank line, aligned left, type the First Author’s name (first the initial of the first name, then the last name). If any of the co-authors have the same affiliation as the first author, add his/her name after an & (or a comma if more names follow). In the following line type the institution details (Name of the institution, City, State/Province, Country and e-mail address of a corresponding author). If there are authors linked to other institutions, after a blank line, repeat this procedure.

The authors name must be in Times New Roman, regular, and font size 12. The institution details must be in Times New Roman, *italic*, and font size 10.

3 ABSTRACT

The abstract should start after leaving eight blank lines. Type the text of the abstract in one paragraph, after a space behind the word abstract and colon, with a maximum of 250 words in Times New Roman, regular, font size 12, single-spaced, and justified. After leaving one blank line, type KEY WORDS: (capital letters, Times New Roman, font size 12), followed by a maximum of five (5) key words separated by commas. Only the first letter of the first key word should be capitalized.

4 THE TEXT

The main body of the paper follows the key words, after two blank lines (i.e., two blank lines between the first heading and the key words). The body text should be typed in Times New Roman, font size 12 and justified. The first line of the paragraphs should be indented 5 mm except the paragraphs that follow heading or subheading (i.e., the first line of the paragraphs that follow heading or subheading should not be indented). Never use bold and never underline any body text.

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The headings are in capital letters, Times New Roman, font size 12. Subheadings are in title letters Times New Roman, font size 12. The headings and subheadings must be aligned left and should not be indented.

Leave two blank lines before and one after the heading. There should be one (1) blank line before and after the subheadings. All headings and subheadings must be numbered.

If a heading or subheading falls at the bottom of a page it should be transferred to the top of the next page.

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Figures, line drawings, photographs, tables should be placed in the appropriate location, aligned centre, and positioned close to the citation in the document. They should have a caption, Times New Roman font size 12, with a colon dividing the figure number and the title (Figure 1: Material properties) and should be numbered consecutively, e.g. Figure 1, Figure 2, Table 1, and Table 2.

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At the end of the paper, list all references with the last name of the first author in alphabetical order, underneath the heading REFERENCES, as in the example. The title of the referred publication should be in italic while the rest of the reference description should be in regular letters. References should be typed in Times New Roman font size 12. citation standard ISO 690.

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