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UTILISATION OF RESEARCH AND DEVELOPMENT RESULTS ON PROFESSIONAL FIELD OF RAILWAY INFRASTRUCTURE

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SUMMARY

The activity and the results which are achieved in research and development topics are important quality characteristics of the individual and institutional achievement in the higher education. The customer's and researcher's interest is common to bring new knowledge into being. The resulting product means technological and economic benefits for customer at the same time brings income and new knowledge for the university, which can be built into the financing of the education. This article deals with the research and development activity of Széchenyi István University briefly, afterwards presents a case of a rail failure phenomenon, the way of understanding and numerical characterisation.

1. ABOUT THE RESEARCH AND DEVELOPMENT WORK IN GENERAL

The research and development activity intents to expand the existing knowledge and to work-out new applications. It covers the basic research, the applied research and the experimental development. Activity of Széchenyi István University on field of railway infrastructure can be classified as applied industrial research, because the acquisition of new knowledge always happens in favour of practical purposes.

The scientific research activity is a complex task and its planning needs great care. First step is to draft the goal and to specify the desired scientific knowledge. This is – might be – only the start, because during the work difficulties, new circumstances can occur, which can even modify the original research programme significantly. The researcher has to realize that the reach of desired result can be encumbered by scientific uncertainties. Alternatives have to be prepared in the planning period of the work, which enable – despite the difficulties and after their reconstitution – to reach the goal.

Professional and practical support of the costumer means very lot in point of view of successful realisation. Research works on field of railway infrastructure in more cases need laboratory tests and trials on track. Costumer's help is indispensable to ensure test samples and performance of test sites. Consultations with costumer are the forums where the evaluation of results and drafting of modified expectations can happen.

It is also important how the result will be utilized, which can be determined significantly by the financial possibilities of costumer. But an unrealized work has always such result, which can be inserted in education or can give new information for specialists in publications.

Our research works in the last forty years were focused on targets as below:

- discovery of a not well known reason of a phenomenon, detection of laws between cause and effect,
- creating new method, solution, equipment and confirmation of their compliance.

Remarkable research and development works in the last ten years were:

- Determining of force impacts generated by railway track and bridge interaction, computer modelling, revision of dimensioning prescriptions (2009-2010)
- Using of geogrids to stabilize railway ballast bed (2009-2011)
- Complex investigation of possibilities of excess cost reduction are caused by velocity reduction on electrified tracks (2011-2012)
- Development of a new method for track transition on railway bridges (2011-2012)
- Management of rail head defaults, development of technical requirements taking economical aspects into account (2013-2014)
- Development of requirements of transition sections between railway bridge and track with complex approach (2015-2017).

In the next chapters one of the most interesting works will be introduced.

2. CHARACTERISTICS OF RAIL HEAD RCF CRACKS AND DESCRIPTION OF PHENOMENON

2.1. Changes in rail material

A failure which is classified as Rolling Contact Fatigue (RCF) spread in large quantities in the early 2000 on the network of Hungarian Railways. This is Head Check (HC) failure, which appears in rail head at gauge corner in groups (see Fig 1). These phenomenon is dangerous because the cracks penetrate into the rail cross section deeper and deeper. At first surface breaks evolve and later, in more serious case, cross section fracture can be occurred.

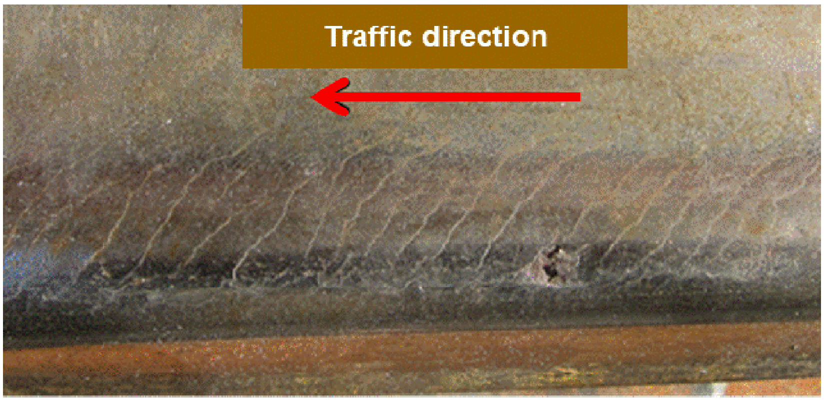

Fig. 1: Rail head hair-cracks, shaped „S"

Failures occur mostly in outer rail of superelevated curves with radius 400 - 3000 m and in switches at curved stock rail and at switch diamond. Centrifugal force at wheel running in curve is the reason that the contact area on the outer rail head shifts into the direction of the gauge corner. Very high compressive and tangential stresses occur on this contact area and a cold-worked layer made by wheels has been formed with a thickness of 0.4-1.2 mm. In this layer the hardness can be even 1.5 times higher than in normal steel structure (see Fig. 2).

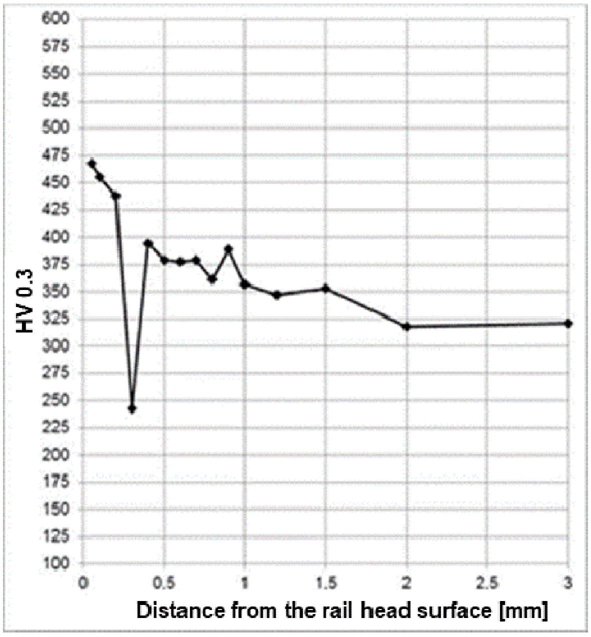

Fig. 2: Change in hardness from the surface toward the rail head interior

The metal structure changes in the hardened layer (see Fig. 3). The deformation ability of the material runs out and it leads to appearance of cracks. The crack runs ahead deeper and deeper into the rail head, follows the elongation direction of the grains.

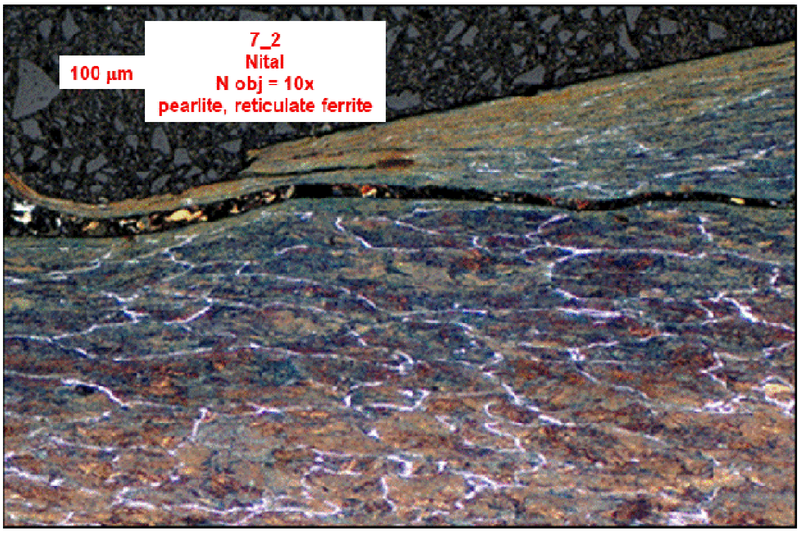

Fig. 3: Deformed material structure and the crack

2.2 Cracks' geometry

Microscope recordings, which are made in cross and longitudinal direction of the rail head, enable to determine the cracks' length, depth, width, penetration angle, change of running direction and the distance between adjacent cracks.

Cracks form in space irregular, overlapping surfaces, as it is shown in Fig. 5, which was made by computer conversation of YXLON Modular industrial CT system recording. In the upper left corner of the figure the original position of the investigated specimen can be seen.

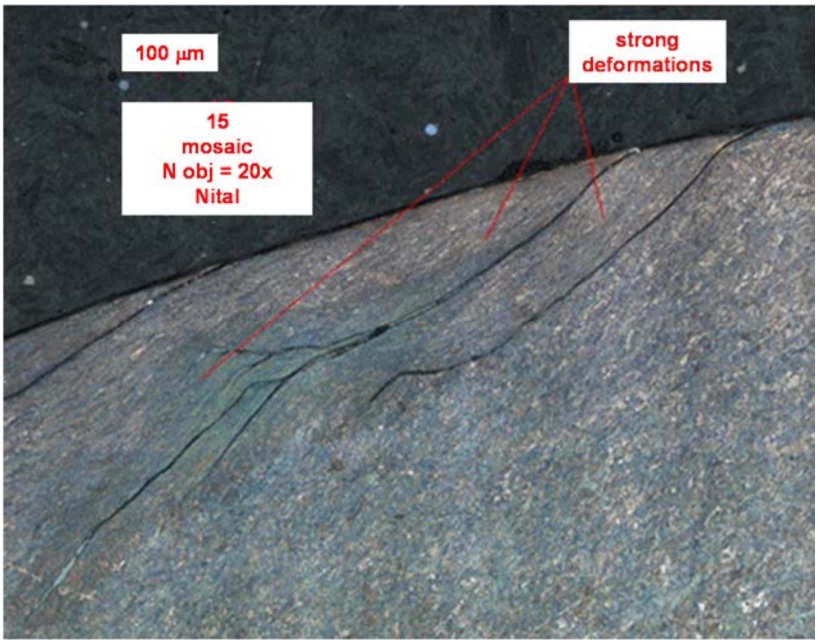

Fig. 4: Cracks running in rail head's cross section

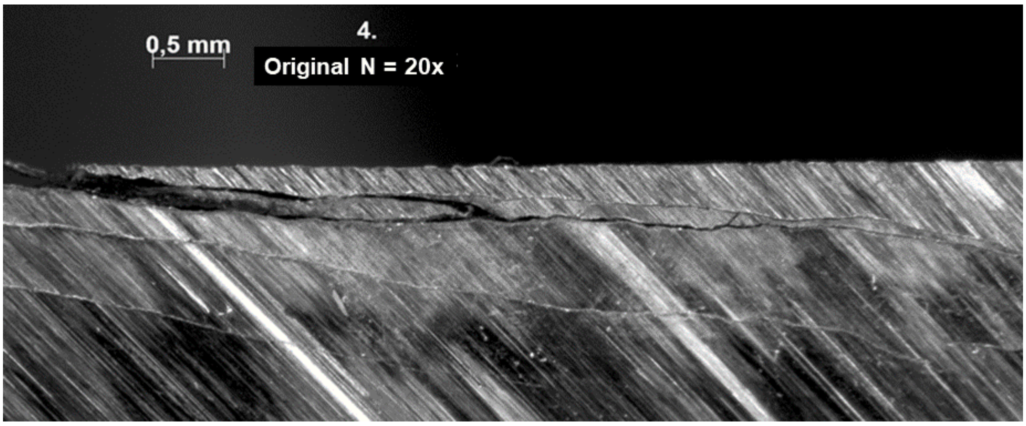

Fig. 5: Cracks in rail head longitudinal section

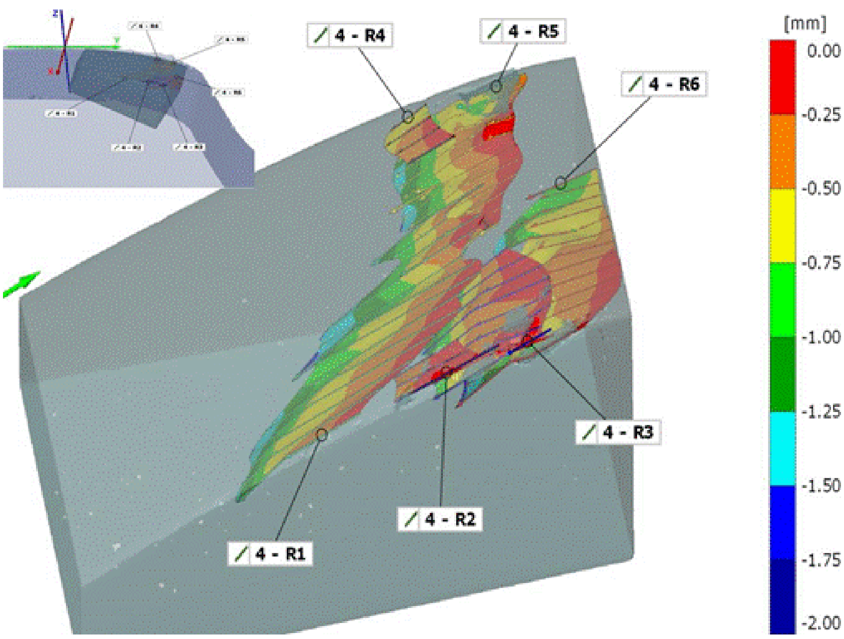

Fig. 6: The position of crack planes in space and the depth values

3. MECHANICAL MEASUREMENT OF RAIL HEAD CRACKS AND EVALUATION OF MEASURED DATA

For successful treatment of rail head hair-cracks the followings have to be solved:

- recognition of failure,
- determination of geometrical characteristics of the cracks (area, intensity of cracks, penetration grade and depth),
- failure measurement in high quantity, data recording and processing,
- numerical characterization of the damage,
- development of tolerance system to classification of the damages,
- determination of the work intervention (method, location, date),
- execution of intervention, checking of efficiency.

A rail diagnostic train (SDS) and a rail diagnostic measuring car (FMK 008), operated by company MÁV KFV Kft., perform the measurement of HC failures in railway network, working on principle of the eddy-currant detection. During the measurement 4-4 probes detect the rail head cracks on both rails. (For measurement of turnouts are available manual devices made by Rohmann GmbH or Metalelektro Kft. respectively).

Data for rails provided by diagnostic train and car are:

- calculated damage depth per meter, per probe in range of 0.01 – 3 mm,
- quantity of cracks per meter and per probe.

At the end we can get per rail the maximum calculated damage depth and the maximum quantity of cracks, provided by office system for the given classification length (1 m or 20 m).

With the help of further processing of data, which are integrated in an Excel chart we can characterize the condition in time of measurement. The change of the condition can be presented by combined processing of data, were collected in consecutive time. Figure 7 shows the damage depths as a function of track sectioning, in case of a curve with 1000 m radius and 120 mm superelevation, for right (outer) rail. According to executed five measurements the condition deteriorated vigorously between April 2014 and April 2016, and the condition was improved very significantly by rail grinding in December 2016.

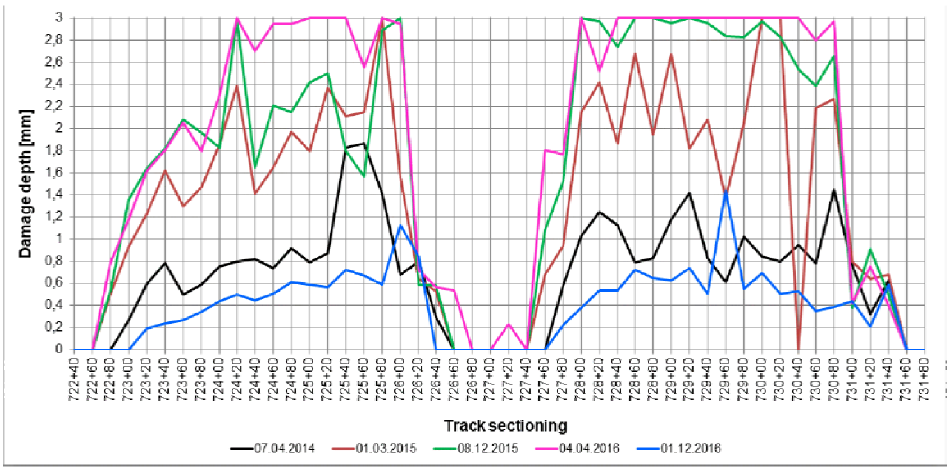

Fig. 7: Damage depths in the course of five measurements as a function of track sectioning

In Fig. 7 the changes can be realized but Fig. 8 is more expressive, where distribution curves of damage depths can be seen in different time. The shifting of four curves (04.07.2014; 01.03.2015; 08.12.2015; 04.04.2016) marks the condition deterioration. Measurements in March and December 2015 and in April 2016 show how the rate of damage depth magnitude of 3 mm or larger grew first from 6% to 11%, after to 40%. (Feature of measurement that the larger values than 3 mm are classified also into category of 3 mm.)

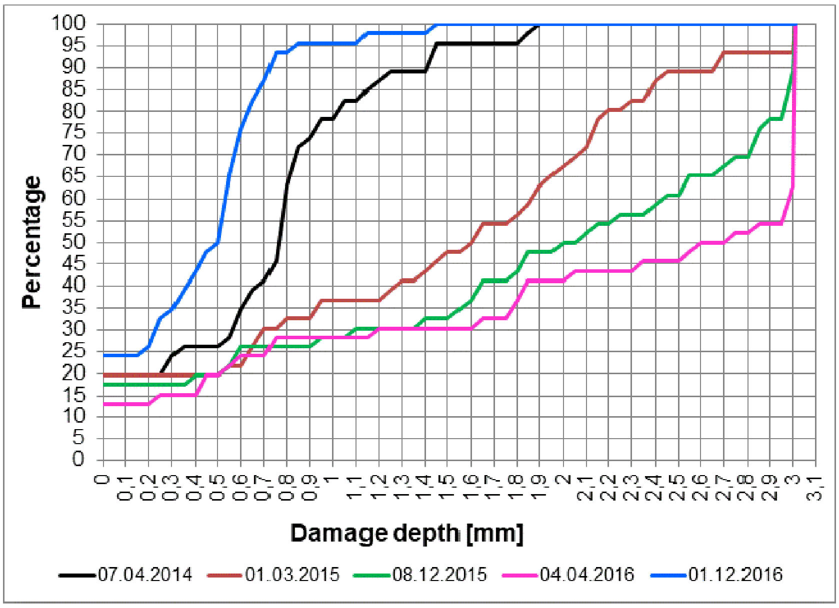

Fig. 8: Change of deterioration curves of damage depths

The position change of distribution curves and together with it the deterioration of condition can be characterized by so called shape numbers. Shape number is secondary moment of the distribution curve onto the vertical axis. Only one shape number belongs to one distribution curve and vice versa, so it can suit for numerical characterization. In Fig. 9 the change of shape numbers of the damage depths can be seen from the first measurement (07.04.2014) as a function of the time. The deterioration in the first three intervals can be considered linear.

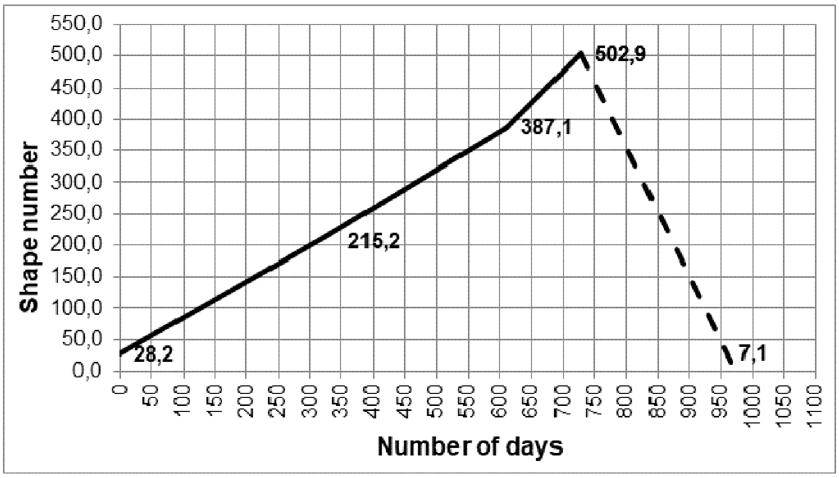

Fig. 9: The change of shape numbers of damage depths as a function of time

Characterization of efficiency of the executed work can be done by EEW values, as follows:

$$EEW = \frac{DD_{before} - DD_{after}}{DD_{before}} \cdot 100 \text{ [\%]}, \text{ where}$$

EEW = efficiency of executed work (e.g. rail grinding) characterized by damage depth values,
DD_{before} = damage depth value before the executed work,
DD_{after} = damage depth value after the executed work.

4. DESIGN OF RAIL TREATMENT

Evaluation of measurement data of HC failures is not enough to design rail treatment further characteristics has to be known as well:

- corrugations on rails,
- vertical and lateral wear of rail head,
- data of equivalent conicity.

In frame of research and development work, in co-work with expert of company MÁV KFV Kft., a new module was born for computer planning system PATER with which the design of complex rail head treatment can be executed. This module takes into account all ageing phenomena, which need either rail treatment or rail replacement ensuring a work design with economical optimal.

Computer program describes the deterioration degree, namely the need of rail treatment or replacement according to a unified method with help of scores, which are calculated from data of different measurements (rail head wears, equivalent conicity). Condition of the given track section can be characterized numerically by simple addition of the single scores. At a certain added score – taking to account the technical and economic circumstances – can be fixed on the limit, above it the rail treatment or replacement has to be executed in a given time. Program works with two types of scores:

- “T” rail treatment score, which means the necessity rate of rail grinding / spreading / milling,
- “R” rail replacement score, which means the necessity rate of rail replacement.

Fig. 10 shows the final proposal for an examined double tracked railway line between kilometre sectioning 65 – 80 and for four rails. Where the rail replacement curve runs above the curve of rail treatment, rail replacement is suggested.

5. INTERVENTION POSSIBILITIES

There are two ways to prevent the evolution of HC failures or to decelerate measurably the cracks’ development. The first way is the selection of rail steel grade, namely construction of rails with higher head hardness, and the second way is using so called AntiHeadCheck (AHC) rail profile.

According to technically and economically coordinated practice rail maintenance has three ways to manage the rail damages:

- preventive rail grinding in case of new constructions and renewals,
- cyclic rail processing (generally rail grinding) with network approach,
- repair works on shorter track sections to prevent the heavy damage (e.g. rail milling).

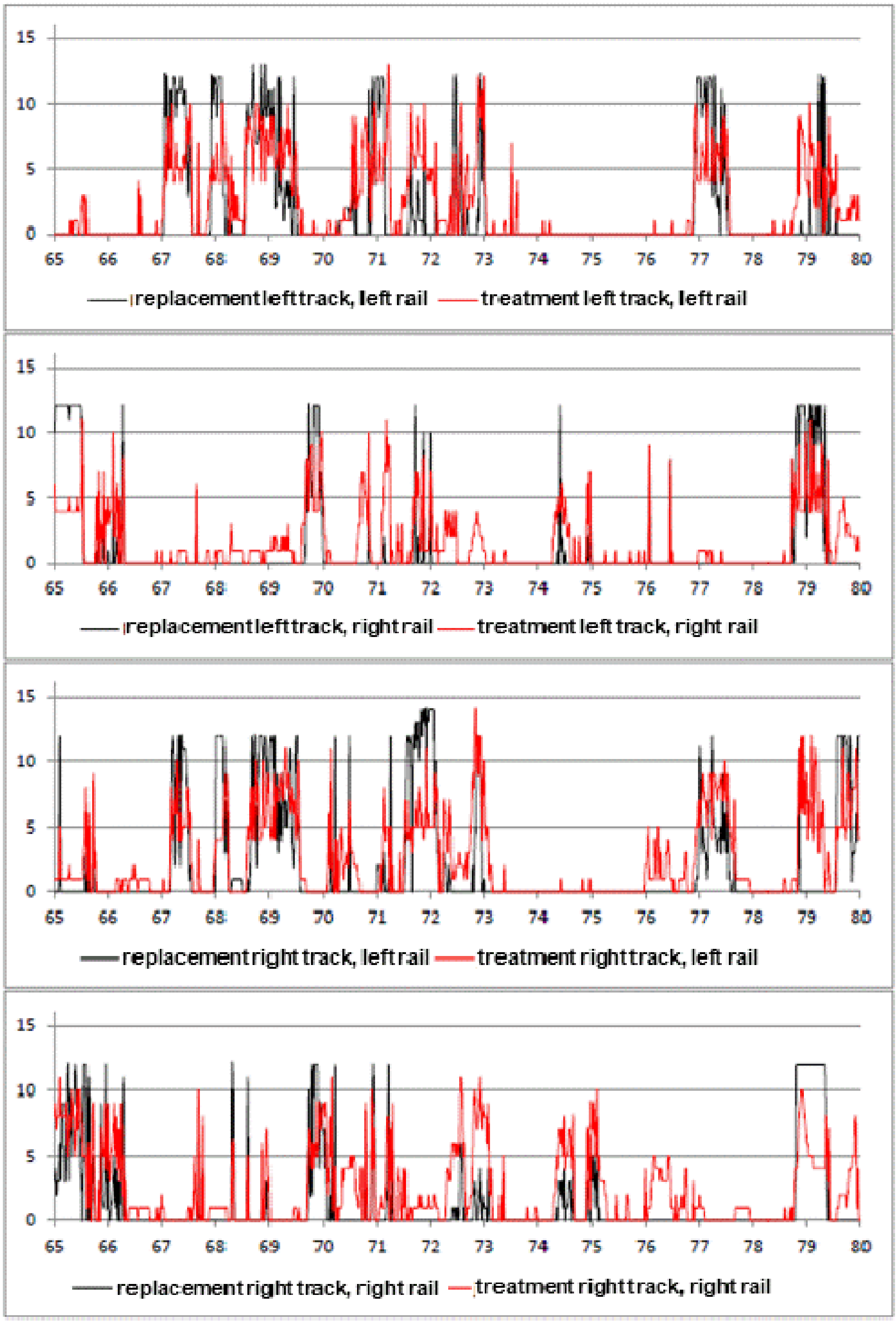

Fig. 10: Suggested track sections for rail treatment or replacement

6. CONCLUSIONS

Analysis of the results of regular rail condition measurements since 2014 on Hungarian railway network shows that the situation in point of rail head hair-cracks is manageable only with large financial investments. Executed intensive works in last years have decreased the number of speed reduces significantly, but a longer-term transitional strategy is necessary for years. This strategy

- eliminates the rail HC cracks, which are dangerous to traffic safety (correctional rail grinding or rail replacement),

- begins the management of a system, which is correct technically and favourable economically (e.g. execution of preventive rail grinding on renewed tracks, where appearance of HC failure is expected and preparation of cyclic rail grinding by more year long design),
- at the end converts to the correct rail treatment strategy on whole network.

It has to be emphasized that a repair effort, which is non-competitive with the deterioration speed, can't bring a favourable change. Therefore the strategy mustn't be established to the available money sources, but it is necessary to ensure financing to the conditions of an economical strategy which can come into being among the given circumstances.

7. ACKNOWLEDGMENTS

Author expresses his thanks for company MÁV Zrt. for financing of research and development work and for railway experts, who professionally supported the project.

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LONG SPAN BRIDGES

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SUMMARY

This paper gives an overview of the evolution of long span bridges, suspension bridges and cable-stayed bridges. It explains why more and more long span bridges are erected around the world, specially in China. The level of design live loads is discussed, and some possible future evolutions are evoked.

Keywords: long span bridges, suspension bridges, cable-stayed bridges, hybrid solutions, traffic loads.

1. HISTORY

1.1. Suspension bridges

Tab. 1, below, evidences that the record span of suspension bridges has been multiplied by about two in a bit less then than a century, in a very limited number of steps. The two major steps came with the Golden Gate Bridge in 1937, and the Akashi Kaikio Bridge in 1998, which is still the world record.

Tab. 1: World record of suspension bridges.

Name of the bridge	Main span (m)	Date of completion	Type	Country
George Washington Bridge	1 067	1931	Two level truss	United states of America
Golden Gate Bridge	1 280	1937	Two level truss	United states of America
Verrazano Bridge	1 298	1964	Two level truss	United states of America
Humber Bridge	1 410	1981	Streamlined box-girder	United Kingdom
Akashi Kaikyo Bridge	1 991	1998	Two level truss	Japan

Fig. 1: The George Washington Bridge

Fig. 2: The Golden Gate Bridge

Fig. 3: The Akashi Kaikyo Bridge

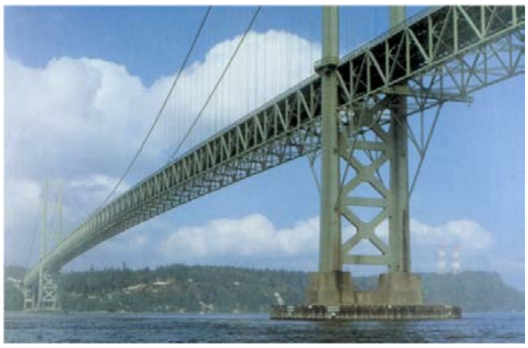

Fig. 4: A classical truss: The New Tacoma Bridge

But this table does not give a good view of the technical evolution of suspension bridges: except for the Humber Bridge, all these bridges take the shape of a two level truss.

In fact a technical revolution took place in the sixties with the first suspension bridges designed with a streamlined deck: the Severn bridge (998 metres in 1966), the first Bosphorus Bridge (1 074 metres in 1973) and the Humber Bridge (1 410 metres in 1981), all designed by Freeman Fox and Partners.

As we shall see later, the design of long span bridges is for a very large part governed by wind effects, aerodynamic stability and resistance to extreme turbulent winds. For longer and longer spans, wind forces produced on decks having the shape of a two level truss become larger and larger, calling for bigger and bigger, heavier and heavier trusses.

This revolution – anticipated by Fritz Leonhardt – has consisted in changing the philosophy: in place of designing structures capable of resisting to larger and larger wind forces, the idea has been to give to the deck a shape which reduces wind forces. This led to the development of streamlined box-girders which, in addition, have a high torsional rigidity and are naturally extremely stable aerodynamically.

Tab. 2: Suspension bridges. The longest spans in 2018

Rank	Name of the bridge	Main span (m)	Date of completion	Deck type	Place	Country
	Canakkale Bridge	2 023	2023	Streamlined box-girder	Canakkale straits	Turkey
1	Akashi Kaikyo Bridge	1 991	1998	Two level truss	Kobe	Japan
2	Xihoumen Bridge	1 650	2009	Streamlined box-girder	Jintang Island	China
3	Storebelt Bridge	1 624	1998	Streamlined box-girder	Sprogo Island	Denmark
4	Osmangazi Bridge	1 550	2016	Streamlined box-girder	Izmit bay	Turkey
5	Yi Sunsin Bridge	1 545	2012	Streamlined box-girder	Gwanguang	Korea (South)
6	Runyang Bridge	1 490	2005	Streamlined box-girder	Yangzhou	China
7	Fourth Nankin Bridge	1 418	2012	Streamlined box-girder	Nankin	China
8	Humber Bridge	1 410	1981	Streamlined box-girder	Barton on Humber	United Kingdom
9	Jiangyn Bridge	1 385	1999	Streamlined box-girder	Jiangyin	China
10	Tsing Ma Bridge	1 377	1997	Road and railway. Two level truss	Hong Kong	Hong Kong
11	Hardanger Bridge	1 310	2013	Streamlined box-girder	Brimmes	Norway
12	Verrazano Bridge	1 298	1964	Two level truss	New York	United States of America
13	Golden Gate Bridge	1 280	1937	Two level truss	San Francisco	United States of America

Tab. 2, above, gives the list of the 13 longer suspended spans in the world last year. Only two recent ones take the shape of a two level truss: the Akashi Kaikio Bridge, which is the last example of such trusses for road bridges, and the Tsin Ma Bridge in Hong Kong which carries a railway line at the lower level of the truss.

Clearly, today road bridges are always designed with a streamlined deck. As we shall see later, this is also possible for bridges carrying in the same time roadways and railways.

We can also note that the number of bridges with very long spans is rather limited, but more and more bridges are erected with a main span between 1 000 and 1 400 metres (22 in the table which can be found on the internet).

1.2. Cable-stayed bridges

The situation is very different for cable-stayed bridges. As shown by Tab. 3 below, the world record passed from 404 to 1 104 metres in 38 years only.

Tab. 3: World record of cable-stayed bridges

Name of the bridge	Main Span (m)	Date of completion	Material	Country
Saint-Nazaire Bridge	404	1974	Steel orthotropic	France
Carlos Fernandez Casado Bridge (Barrios de Luna)	440	1983	Pre-stressed concrete	Spain
Alex Fraser Bridge (Annacis Island)	465	1986	Composite	Canada
Ikuchi Bridge	490	1991	Steel orthotropic	Japan
Skarnsund Bridge	530	1991	Pre-stressed concrete	Norway
Yangpu Bridge	605	1993	Composite	China
Normandie Bridge	856	1995	Steel orthotropic	France
Tatara Bridge	890	1999	Steel orthotropic	Japan
Sutong Bridge	1 088	2008	Steel orthotropic	China
Russky Island Bridge	1 104	2012	Steel orthotropic	Russia

Fig. 5: The Skarnsund Bridge

Fig. 6: The Normandie Bridge

The main reason is that engineers have not immediately understood the effective capacity of cable-stayed bridges. It is easy to see it from the fact that the world record passed twice from a steel orthotropic deck, to a pre-stressed concrete deck and then to a composite one; as self-weights are (very roughly) in proportion of 1 for a steel orthotropic deck, 2 for a composite one and 3 for a pre-stressed concrete one, the historical evolution shows that the world records in the seventies and eighties were very far from the limit. It is now considered, for economical reasons, that the limit is about 500-550 metres for pre-stressed concrete bridges, and about 700 metres for composite ones. Above 700 metres, cable-stayed bridges must have a steel orthotropic deck.

Another example is the fact that in Germany it has been possible to erect three bridges with main spans of more than 300 metres with a unique tower (Köln Severin bridge, 302 metres in 1959; Düsseldorf Kniebrücke, 320 metres in 1969; and Düsseldorf Flehe Bridge, 368 metres in 1979), equivalent to bridges with two towers having a main span between 550 and 650 metres.

Tab. 4, below, gives the list of the nine longer cable-stayed spans in the world today. We can note the number of recently erected bridges, specially in China.

Tab. 4: Cable-stayed bridges – The longest spans in 2018

Rank	Name of the bridge	Main span (m)	Date of completion	Place	Country
1	Russky Island Bridge	1 104	2012	Vladivostok	Russia
2	Sutong Bridge	1 088	2008	Suzhou-Nantong	China
3	Stonecutters Bridge	1 018	2009	Hong Kong	Hong Kong
4	Edong Bridge	926	2010	Huangshi	China
5	Tatara Bridge	890	1999	Omishima	Japan
6	Normandie bridge	856	1995	Le Havre	France
7	Juijang Bridge	818	2013	Jinjang	China
8	Jingyne Bridge	816	2010	Jingzhou	China
9	Incheon Bridge	800	2009	Incheon	Korea (South)

Fig. 7: The Tatara Bridge

Fig. 8: The Russki Island Bridge

But it does not give a good idea of the explosive development of cable-stayed bridges during the last 20 years. For a clear evidence, the list of the longer spans in the world in 2018, which could be found on the internet, shows that the past world records were very far in the list:

– the Saint-Nazaire Bridge was ranked 102;
– the Carlos Fernandez Casado Bridge was ranked 70;
– the Alex Frazer Bridge was ranked 55;
– he Ikuchi Bridge was ranked 41;
– and the Yangpu Bridge was ranked 22.

There are 30 bridges in the world with a main span between 500 and 600 metres, 68 bridges with a main span between 400 and 600 metres.

1.3. Hybrid solutions

1.3.1. In the 19th century several bridges – the most famous being the Brooklyn Bridges – have been erected with a hybrid design, associating main suspension cables and hangers with stay cables in the areas close to the towers (on a distance equal to one fourth or one third of the main span).

This solution has been adopted to produce more rigidity, the decks being too flexible at these times to resist high loads and wind effects.

It has been abandoned in the 20th century, the design of stronger decks producing the necessary rigidity.

1.3.2. With Jean-François Klein, we came back to this concept for the design of the Third Bosphorus Bridge (Yavuz Sultan Selim Bridge). Our goal was to design an elegant structure – as requested by the Client – capable of carrying the two tracks of railways which are added to the eight motorway lanes (four in each direction).

In a pure suspension bridge, the passage of heavy trains at quarter spans would produce very large downwards deflections, not acceptable for the train traffic, due to the longitudinal displacement of the main cables. The addition of stay cables produces the necessary rigidity: the train load is transferred to the adjacent tower head, which is itself stabilized by the backstays in rigid side spans on multiple supports.

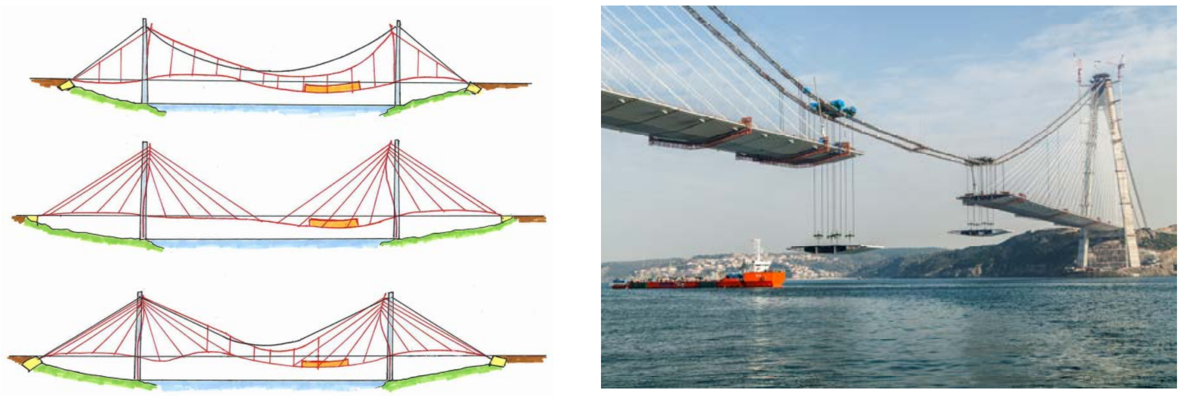

Fig. 9: The Third Bosphorus Bridge

It allows for designing the deck as an elegant streamlined box-girder, in place of a two level truss.

Tab. 5: Modern hybrid structures: bridges with suspension cables and stay cables

Name of the bridge	Main span (m)	Date of completion	Type	Country
Yavuz Sultan Selim Bridge	1 408	2016	Road and railway. Stream lined box-girder	Turkey

Fig. 10: The Third Bosphorus Bridge (Photo ICA)

1.3.3. Of course this solution calls for specific design needs, for an example the need of a transition zone with in the same time stay cables and hangers, to avoid a brutal passage from a rather rigid cable-stayed cantilever to a flexible suspended central part of the span.

2. THE REASONS OF THE EVOLUTION OF SPAN LENGTHS

There are many reasons to explain the multiplication of long span bridges during the last twenty years.

- The first one is very clear: all bridges which were necessary for social and political reasons have been built when this was easy, or possible with the existing methods and means. So that the most difficult bridges to be built had been left.

- Contracting companies are bigger and bigger, and have much larger capacities from a constructional point of view, but also financially. It opens large possibilities, specially when more and more projects are built within Public-Private-Partnerships, due to the more and more limited financial investing capacities of governmental bodies.

- There is a constant progress in materials and technology. For an example, the ultimate tensile stress of strands was equal to 1 770 MPa 30 years ago; it passed to 1 860 MPa; we now use strands with a Guaranteed Ultimate Tensile Stress (GUTS) of 1 960 MPa; and researches are made, with first applications, with a strength of about 2 100 MPa. But this is not enough to explain the observed development of long span bridges.

- Of course the development of the information technology helps developing larger and larger projects, allowing for complex computations and now taking a direct part on site, with geometry control and with the capacity of modelling with the computer all construction equipment and erection steps. But this is only a tool, which helps but cannot do more.

- In our opinion the major reason for such a rapid development of long span bridges, and of the possible erection of bridges in very difficult conditions, is a much better knowledge of natural forces. We can now predict very accurately wind induced effects and earthquake actions on structures, even in very difficult sites and for extreme seismic forces. Even cable vibrations can be more easily predicted – and controlled – than 20 years ago.

3. DESIGN EVOLUTION

The design evolution is rather clear.

3.1. The design of long span bridges is at first governed by wind effects.

It calls for streamlined decks, for aerodynamic stability, but also to reduce the effects of turbulent winds to which the bridge has to resist.

With increasing span lengths, the width of the deck becomes a critical parameter. If the ratio of span to width is too large (we cannot fix a limit, but it becomes necessary to be careful when it is larger than 40), the transverse dynamic displacements produced by turbulent winds can become too large and critical.

This is why it has been proposed, 20 or 30 years ago, to divide the deck into two parallel box-girders joined by a series of strong cross-beams, in order to improve the aerodynamic stability and the resistance to horizontal transverse forces.

The vertical aerodynamic damping on the individual box-girders increases the global torsional damping.

3.2. This solution has been considered for the Messina Straits project, with three box-girders, one on each side for the road traffic, and a central one for railway tracks.

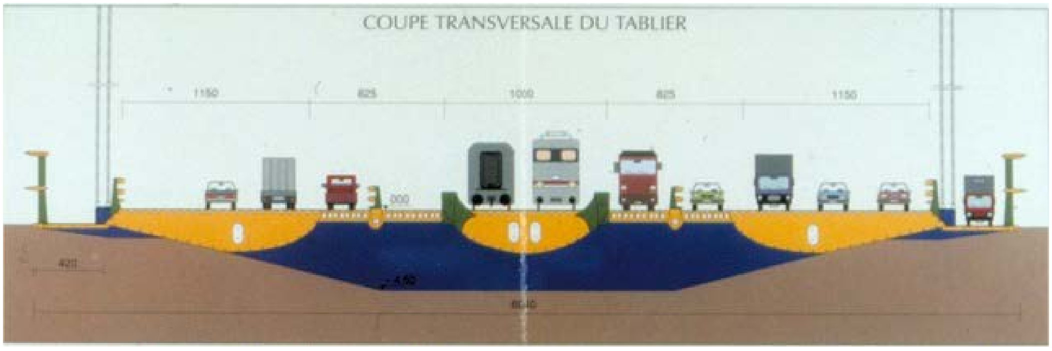

Fig. 11: The cross section of the Messina Bridge Project

It has been adopted for the erection of the Stonecutters cable-stayed bridge in Hong Kong, and – as far as we know – for a suspension bridge in South Korea.

Fig. 12: The Stonecutters Bridge (Photo ARUP)

It will be adopted for the future world record, the Canakkale Bridge, in Turkey, under construction with a main span of 2 023 metres.

3.3. With T-Ingénierie, we took part in the design and built competition, and made specific analyses of this efficient solution. The distance between the two box-girders must be carefully analysed; if it is too large, wind forces on the downwind deck could become unfavourable, the wind being more turbulent after its passage around the upwind deck; if it is too small, the structural effect will be limited.

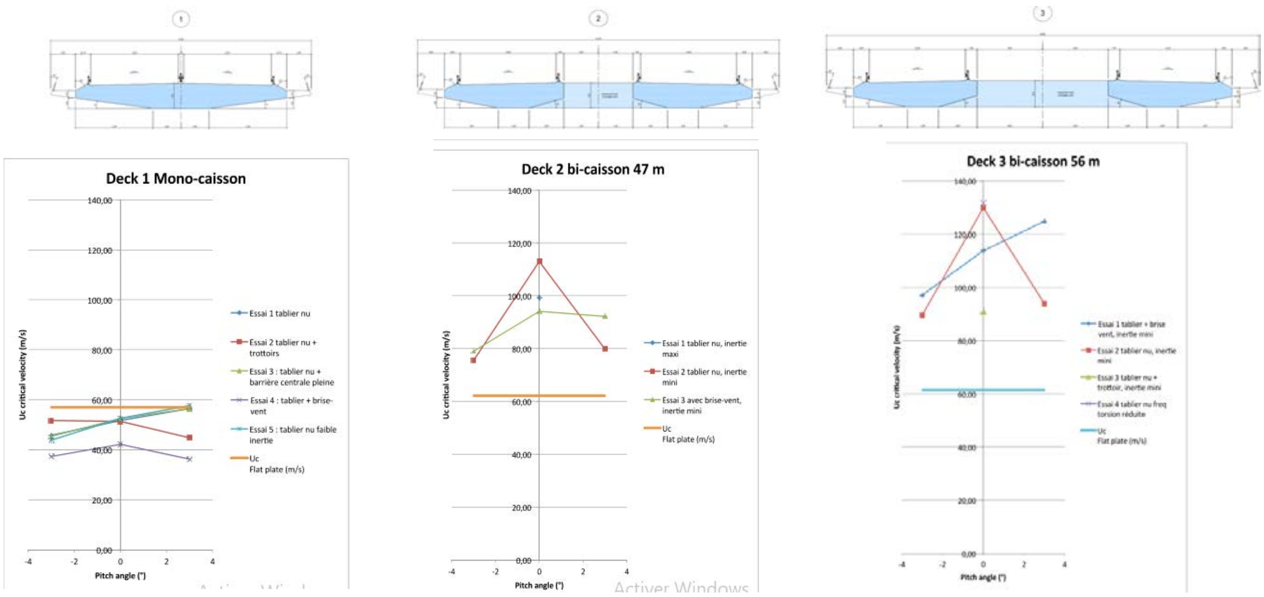

Fig. 13: Effect of the distance between the two box-girders

3.4. We must also evoke the construction of bridges with several successive suspended or cable-stayed spans.

This is the case for the Chiloe Bridge in Chile, with two successive spans of more than 1 000 metres. It calls for a very specific design of the central tower, on the Rocce Molinos, which must be rigid enough to resist the effects of unsymmetrical traffic or wind loads; noting that winds, currents and earthquakes are specially severe in this area.

This is much easier for cable-stayed bridges, even if the design must be adapted to this specific type of bridges to resist unsymmetrical traffic or wind induced loads. The best examples are the Millau Viaduct and more recently the new Forth Crossing, which has the two longer spans in the world – 650 metres – with a composite deck.

Fig. 14: The Millau Viaduct

Fig. 15: The Second Forth Crossing

3.5. We must also evoke the design of curved cable-stayed bridges, even if the span length cannot be very long in this case. A good example is the Térénez Bridge, in French Britany, with a concrete main span of about 280 metres.

Fig. 16: The Térénez Bridge

4. TRAFFIC LOADS

4.1. Long span bridges are extremely sensible to the level of traffic loads, due to their long spans.

We must regret that there is no international standard to fix reasonable values. We consider that, for the definition of traffic loads, existing Codes are not adapted to very long span bridges.

The single exception, as far as we know, is due to the Swedish Authorities which have developed a code (TKBro) fixing much more reasonable values.

It is necessary to make a clear difference between local loads and distributed loads, and to take the span length into account for the definition of widely distributed live loads.

4.2. It is clear that actual local loads, when measured on existing bridges, can be larger than given by Codes since trucks can be much heavier than allowed. But it mainly concerns local effects, such as fatigue effects in thin concrete slabs, in orthotropic decks, or at some specific welding details. Or for the design of floor-beams and troughs.

These overloads cannot be extended to loads distributed on long spans.

4.3. Eurocode loads, for an example, are very heavy; but their application is normally limited at 200 metres. Nevertheless many owners refer to these loads, or even to heavier ones, independently from the span.

This is especially critical for long span cable-stayed bridges, since a high ratio of live loads to self-weights leads to an increased sag in stay cables, reducing their efficiency without any need.

4.4. This is even more critical for bridges carrying railway tracks, when design loads are specified to be those of the UIC (Union Internationale des Chemins de fer), which are very heavy, and are in addition factored by a coefficient which can reach 1.40 to guarantee the possible evolution of the trains weight in the future.

4.6. For the design of the Third Bosphorus Bridge this has been a very critical question. After long discussions, the UIC loads (two tracks) have been factored by 1.33 only, and we have slightly reduced the Turkish road loads which are independent from the span.

4.7. The principles of the analyses performed to fix the definition of road traffic loads (simulation of a jam produced by an accident) is not adapted for very long spans. t lanes.

In addition, it is possible today, for very specific bridges (and this is the case for all very long span bridges) to control the traffic permanently; it is possible to close the bridge in less than ten minutes (as this has been the case when a fire took place on the Rion-Antirion Bridge), and to avoid the constitution of a major jam on the bridge.

It could also be possible to prevent two heavy trains to cross a bridge in the same time.

Modern information technology could help limiting live loads at a reasonable level, allowing for less expensive and more elegant structures.

COST SAVING POTENTIALS BY THE USE OF NEW RAIL STEELS BASED ON LIFE CYCLE COST CONSIDERATIONS

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SUMMARY

Currently the rail steel 400 UHC® HSH® is tested in the Hungarian railway network with the expectation to reduce maintenance and extend rail service life. In this article Life Cycle Costing method is suggested as a tool for product assessment to prove the economic benefits for the Hungarian State Railways - MÁV.

For 400 UHC® HSH® a quantitative economic simulation is demonstrated based on the comprehensive, worldwide track experiences with this rail steel, demonstrating the expected savings for MÁV.

An outlook is provided, discussing the potential of a new Head Check – free rail steel in the MÁV network.

1. THE USE OF LIFE CYCLE COSTING FOR PRODUCT ASSESSMENT

Among European Railway Organizations procurement processes are well established for various products, in particular for railway rails. Usually products are specified according to a certain standard (e.g. EN13674-1:2017 for flat bottom rails), describing the main characteristics of the product, supplemented by additional criteria from the Railway Organization and suppliers are asked to submit their quotation. This public procurement is the most common process for railway projects in Europe.

Despite the fact that more and more organizations start to follow a MEAT principle (most economically advantageous tender) the aforementioned procurement process is widely used and still hinders the market entry of new products because alternative quotes with new technologies that are deviating from the specifications are usually not awarded.

Following the aforementioned procurement process, the only way to introduce new products in a network, is field testing a product followed by a product assessment.

Ultimately these leads to a very important question:

How shall new products or technologies be assessed to the benefit of the railway organizations and finally also to the benefit of the general public?

To answer this question, it is important to understand, that railway infrastructure is characterized by long-term usage. Investment decisions of today will also affect the costs and the budget of future generations. Thus, product life cycles should be as long as possible on the one-hand to reduce depreciation costs but furthermore the product service lives of different track component need to be harmonized, in a way that an efficient renewal of all track components at the same time is possible.

Furthermore maintenance requirements need to be satisfied. Not only shall the maintenance necessities be as low as possible but also quick and safe repair processes must be applicable and reliable and automated inspection technology must be available.

All these and other parameters can be considered in a three-step analysis consisting of
- In-Track testing of new products
- A technical analysis (RAMS) to display Reliability, Availability, Maintainability and Safety parameters of the new product
- A LCC analysis to display the economic benefit, the optimum field of application and the economic impact of different maintenance strategies

Especially among European Railways this Life Cycle Cost analysis has evolved to a decision making tool for the integration of new product, making sure that all costs in a product life cycle are considered and that the investment in new technologies ultimately pay off for the tax payers.

In the words of B. Franklin:

"The Bitterness of poor Quality remains long after the sweetness of low prices are forgotten"

2. METHODOLOGY - LIFE CYCLE COSTING FOR RAILS

As a leading supplier of railway infrastructure technology, voestalpine has continuously developed its products for all kinds of railway applications to reach the longest possible service lives and lowest maintenance needs. The aforementioned terms RAMS (Reliability, Availability, Maintenance and Safety), LCC (Life Cycle Costs), as well as CSR (Corporate Social Responsibility) have thus been key terms that accompanied product development from the early days but have gained more and more importance over recent years.

The basis for these analyses is our long standing in-field testing experiences with currently 112 long-term track test of different rail steels under various conditions. During track testing several degradation mechanisms of rails are measured continuously such as:

- Rail wear
- Corrugation
- Rolling Contact Fatigue defects

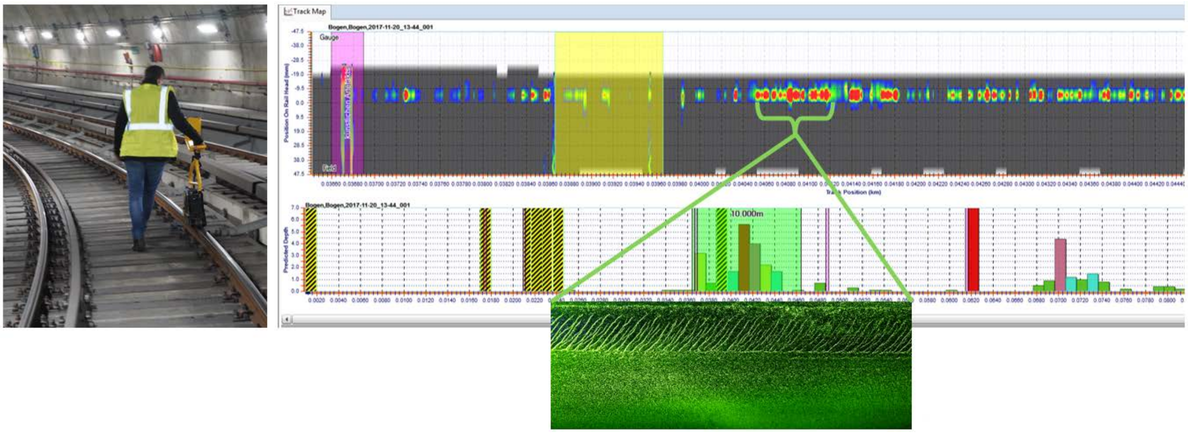
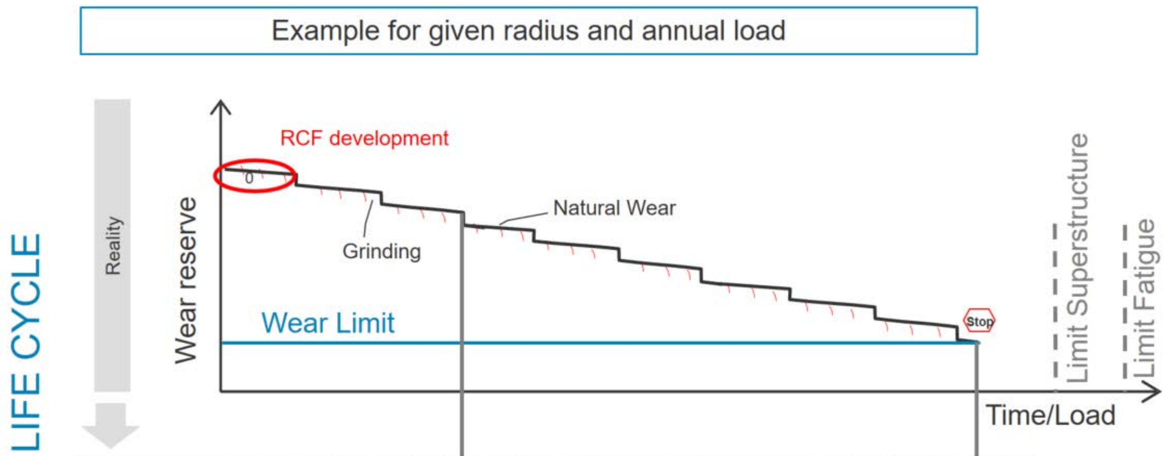


Fig. 1: Example – measurement of Head Checks (RCF) with RSCM-technology

Track testing is usually conducted in close cooperation with the infrastructure operator and when sufficient data is collected the product life cycle can be described. In case of rail, all influence factors that limit service life need to be considered, all maintenance actions as well as additional operational hindrance costs (OHC) arising from non-availability of track.



Year	0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	...
New Layer	1	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	1	...
Grinding	-	-	1	-	1	-	1	-	1	-	1	-	1	-	1	-	1	-	1	-	-	...
OHC	-	-	1	-	1	-	1	-	1	-	1	-	1	-	1	-	1	-	1	-	-	...

Fig. 2: Example - Description of a product Life Cycle based on field data

The Life Cycle Costs comparison between different products is usually done on the basis of equivalent annual costs or on the basis of the Net Present Value mostly considering a fixed time period of 30 to 40 years considering a residual value, when the product service live exceeds the period under consideration.

Furthermore the Pay-back time and the Return on invest can be calculated easily, once the modelling of the life cycles is completed.

3. NEW RAIL STEELS FOR THE HUNGARIAN RAILWAY NETWORK

Together with the Hungarian infrastructure operator Magyar Államvasutak (MÁV) voestalpine is currently introducing 400 UHC® HSH® rail steel to the Hungarian market.

The rail steel 400 UHC® HSH® (specified in EN13674-1:2017 as R400HT) is a pearlitic rail steel originally developed for Heavy Haul railway operation which offers highest resistance against wear and corrugation (two degradation mechanisms that predominantly occur in tight curves). Furthermore it offers an increased resistance against formation of Head Checks which represent the major degradation mechanism in medium to wide curves. Compared to the actual benchmark rail steel R350HT the propagation of these degradation mechanisms is reduced by approximately 50%, making it the ideal rail steel for tight curves with further potential for medium and wide curves.

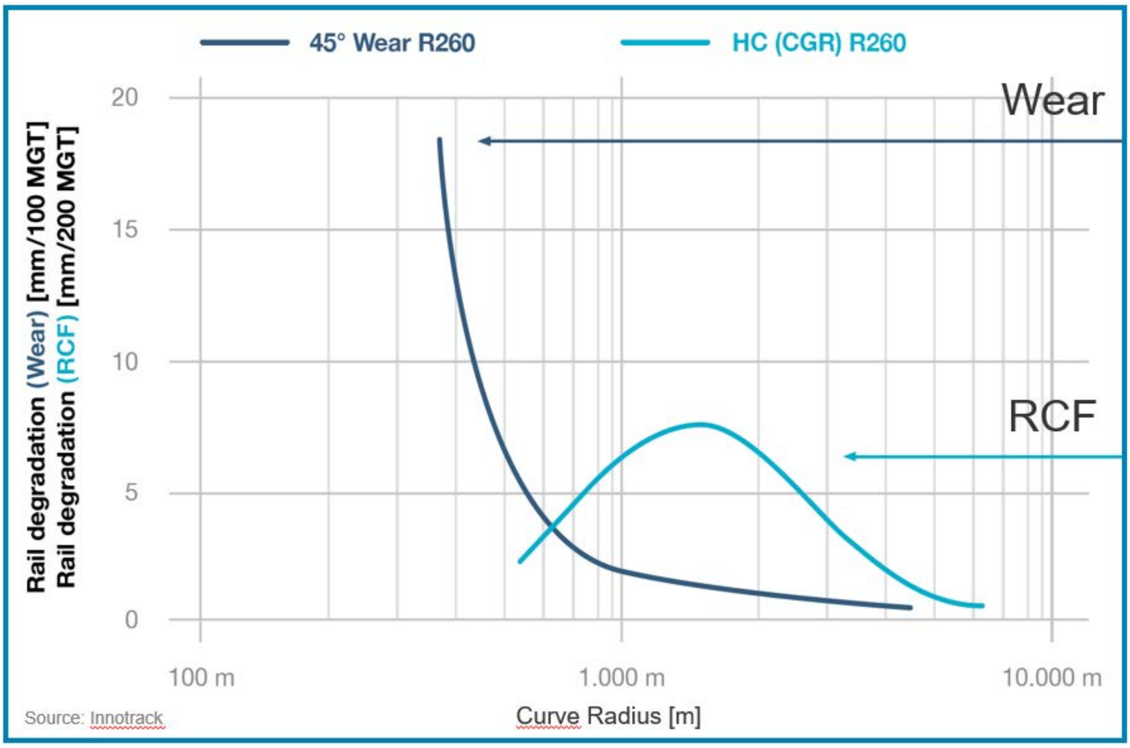


Fig. 3: Degradation mechanisms and their typical area of appearance

4. TRACK RESULTS IN THE HUNGARIAN RAILWAY NETWORK

4.1. Track testing of 400 UHC® HSH®

Besides various track experiences in heavy haul, metro and other mixed traffic lines, the rail steel 400 UHC® HSH® has been tested in a R=300 m Radius curve close to Budapest Kelenföld station since Oct. 2015. The performance of the rails is compared to the rail steel R260 that was implemented in this curve before and has been also implemented in the opposite track of this double track section.

The service life of the rail steel R260 has been only 2.3 years (~ 56 MBGT) due to severe side wear on the high rail – see Fig. 4

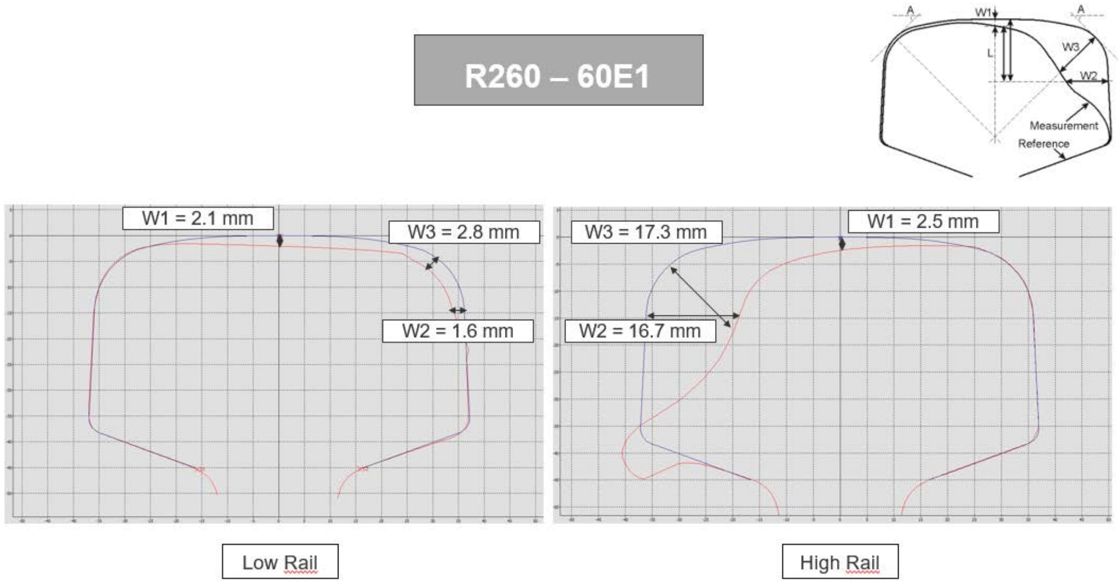


Fig. 4: Wear of R260 rail steel in R=300 m curve after 2.3 years (56 MBGT)

For the 400 UHC® HSH® rails, wear was measured continuously, leading to less than 2 mm of side wear after the same experienced traffic load – see Fig. 5.

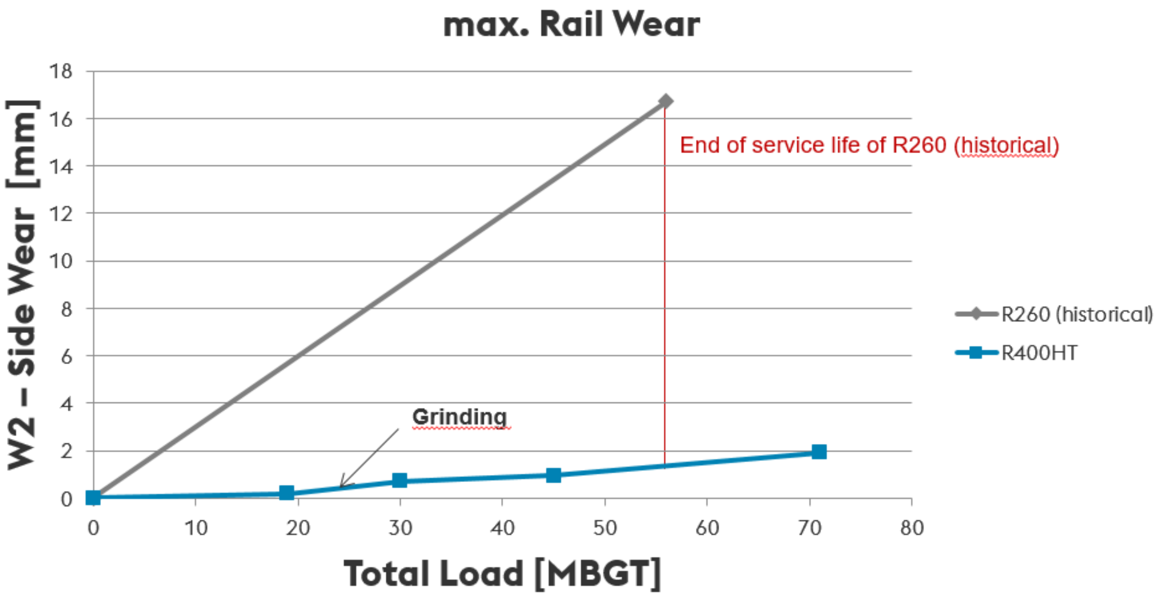


Fig. 5: Comparison of rail side wear of 400 UHC® HSH® (R400HT) and R260 rails

Furthermore a significant reduction of corrugation development was observed when comparing 400 UHC® HSH® rails with R260 rails of the opposite track – see Fig. 6, where measurements of the rail surface at different positions are displayed. R260 already shows significant corrugation of 0.4 mm while corrugation is just beginning after 63 MBGT. For R260 rails, the corrugation has already lead to a degradation of ballast, indicating the necessity of damping, a fact that is not considered in later LCC analyses.

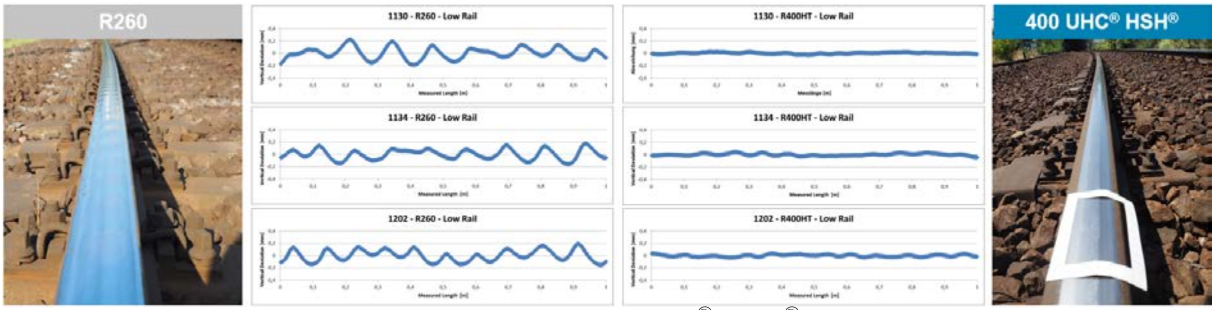


Fig. 6: Depth of corrugation of R260 and 400 UHC® HSH® in comparison – reduction by factor 8

The depth of Head Checks was measured by MÁV with eddy current handheld devices in serveral positions, demonstrating a reduction of the Head Check depth from max. 2.7 mm for R260 to max. 0.6 mm for 400UHC® HSH®. Continous measurement with Eddy current by the MÁV KFV measuring train showed a reduction by factor of 2.7.

Based on the measurements a prognosis for the service life and ideal maintenance cycle of 400 UHC® HSH® rails can be deducted by using usual maintenance intervention limits – see Fig. 7. As can be seen, no rail grinding was conducted for R260 as service life was limited and so short that grinding for corrugation or Head Checks would have been uneconomic.

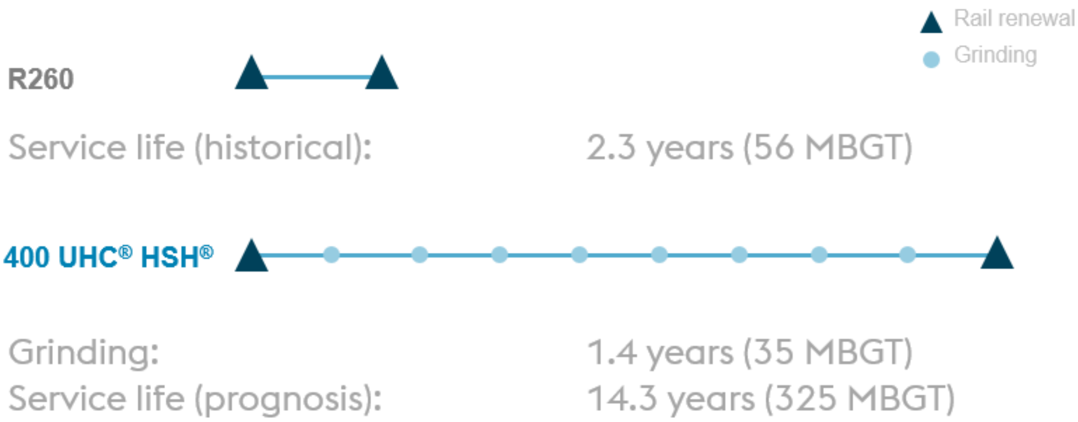


Fig. 7: Modelling of service life cycles of R260 and 400 UHC® HSH®

5. ECONOMICAL POTENTIAL OF NEW RAIL STEELS

5.1. 400 UHC® HSH® - LCC advantage based on track test results

Based on the Life Cycle model market typical costs have been allocated to the respective maintenance action to calculate equivalent annual costs, using dynamic investment appraisal methods (incl. interest rates and inflation).

By applying the methodology for the curve in Budapest Kelenföld, the reduction of equivalent annual costs of 400 UHC® HSH® compared to R260 is in a range of 75 % (see Fig. 8).

The high depreciation costs in case of R260, caused by the short service life, are the main driver for the high Life Cycle Costs in this case.

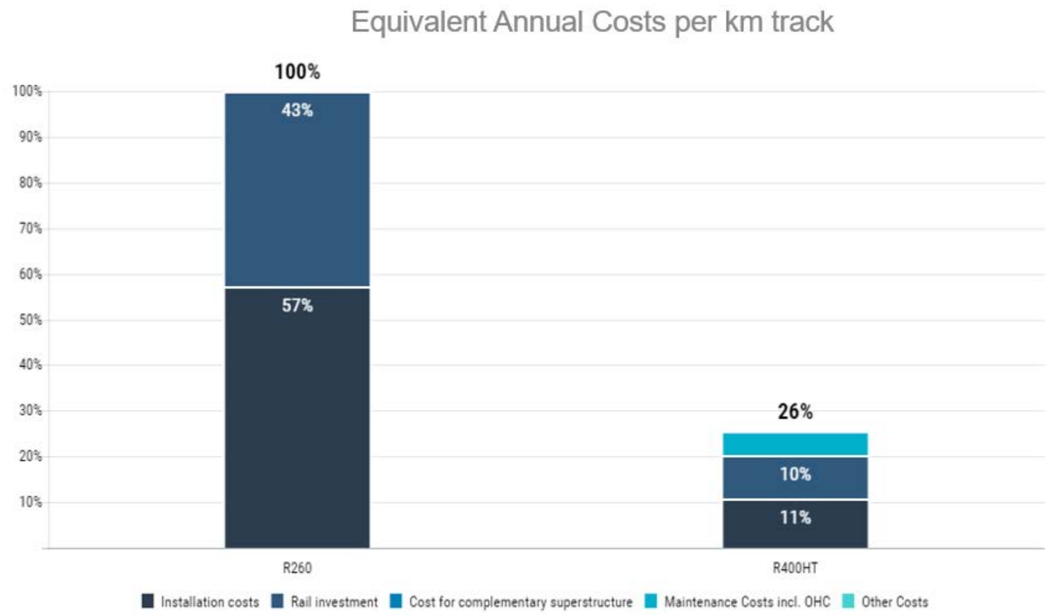


Fig. 8: Equivalent annual costs for R = 300 m curve in Budapest Kelenföld

5.2. LCC potential of new rail steels in the Hungarian core network

5.2.1. 400 UHC® HSH®

Already having gained sufficient experience to appraise the material behaviour of 400 UHC® HSH® rails in different track sections, a LCC analysis for the core network has been conducted to assess the economic impact of using 400 UHC® HSH® rails.

For the analysis track and traffic data has been provided by MÁV containing the most relevant railway tracks of the Hungarian network – see Fig. 9.

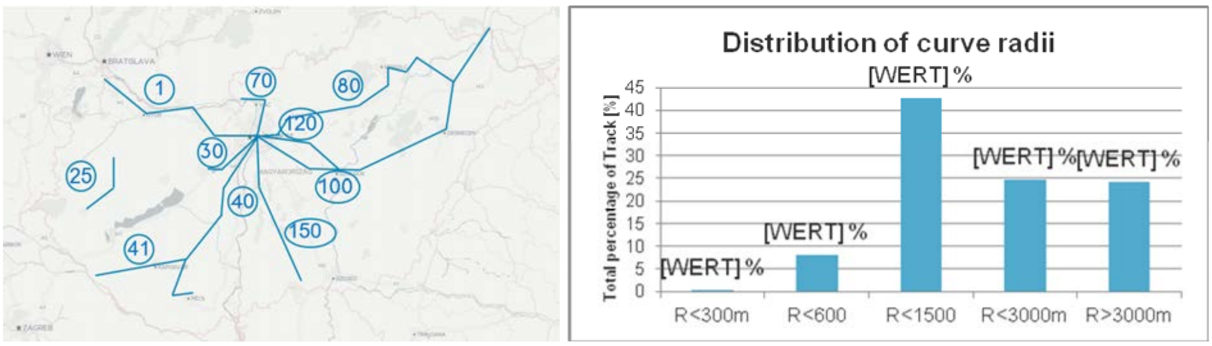


Fig. 9: Core network – basis for LCC Analysis

Comparing 400 UHC® HSH® with the rail steels R350HT (current standard for curves but not yet fully implemented) and R260 (current standard for tangent tracks and still implemented to a large degree in curves) shows a stable ranking of the Life Cycle Costs of the rails, with the lowest LCC for 400 UHC® HSH®.

The cost saving potential by using R400HT rails in curves R<3.000 m in comparison to other rail steels is displayed below:

Tab. 1: Estimated cost savings over 30 years (Net Present Value) by using 400UHC® HSH® in curves

	Compared to R260	Compared to R350HT
Approx. savings by 400 UHC® HSH® in 30 years	**144 Mio. €** **47 Bn. HUF**	**45 Mio €** **15 Bn. HUF**

Tab. 2: Estimated annual savings (Equivalent Annual Costs) by using 400UHC® HSH® in curves

	Compared to R260	Compared to R350HT
Approx. annual savings by using 400 UHC® HSH®	**5.5 Mio. €** **1.7 Bn. HUF**	**2 Mio €** **0.6 Bn. HUF**

5.2.2. The potential of a Head Check Free Rail

Since the last decade voestalpine Schienen has spent considerable efforts on developing a rail steel free of Head Checks. For a network as shown in Fig. 9, a Head Check free rail would bring significant advantages, as 68 % of the track consists of curves between 600 and 3,000 m Radius. Curves, where Head Checks can be considered as the most maintenance causing degradation mechanism.

Considering the actual rail grinding budget of MÁV is in the range of Millions of Euros, it can be concluded that a majority of this amount can be saved, when Head Checks in curves between 600 and 3,000 m Radius could be eliminated.

Furthermore it is a fact that actual grinding capacity is not yet sufficient, leading to a high percentage of speed restrictions in the Hungarian network due to degraded rails. By using a Head Check free rail, these problem could be solved ultimately leading to a significantly lower number of speed restrictions.

From a technical perspective the reduced metal removal from reduced grinding of a Head Check free rail will also lead to a longer rail service life compared to other rail steels, as artificial wear is reduced and natural wear is low in the ideal are of application.

Ultimately the low maintenance concept leads to highest track availability and thus to lowest operational hindrance costs for rail replacement buses, delays or cancellations.

6. CONCLUSIONS

The use of 400 UHC® HSH® rail steel carries vast economic potential in the Hungarian railway network. For 400 UHC® HSH® and also for a future Head Check-free rail steel the reduction of rail maintenance will not only significantly contribute to cost savings as demonstrated in chapter 5 but will further reduce maintenance and corrugation related noise and vibrations as well as increase the availability of the whole track system.

By jointly approving new rail steels in track, MÁV and voestalpine are setting steps towards a more efficient and more economic railway infrastructure.

7. ACKNOWLEDGEMENTS

voestalpine Schienen is grateful for the fruitful partnership with MÁV that has established over the recent years. Special thanks goes to the responsible technicians of MÁV trying to further optimize the Hungarian Railway Infrastructure:

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Marosi Ákos
Nagy Istvan

EVOLUTION OF BRIDGE CONSTRUCTION - NON-METALLIC BRIDGES

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SUMMARY

Concrete engineers and researchers have been developed durable reinforce concrete technologies for long time. However, we have not reached the goal which gives us a perfect technology against deterioration of reinforced concrete. Basically concrete itself is high durable material. And we recognize this fact when we see Roman concrete structures are still working now. The technologies described in this paper are the challenge to derive the solution against deterioration of reinforced concrete. This research and development has been taking for about 30 years. Then a non-metallic highway bridge is under design and will be built in 2020 with the key technologies of aramid fibre tendons, fibre reinforced concrete and butterfly web.

1. INTRODUCTION

Concrete is innately an extremely durable construction material. This is clearly evident to anyone looking at the Pantheon in Rome (Fig. 1), which was built 2000 years ago but is still structurally sound today. However, the Pantheon and other long-lasting concrete structures were made of plain unreinforced concrete. This limited their forms to arches and domes, which are not subject to tensile forces. That changed in 1867, when the French landscape gardener Joseph Monier came up with the idea of strengthening plant pots with wires (Fig. 2), marking the beginning of the history of reinforced concrete. Through the lens of concrete's long history, a development that occurred merely 150 years ago is still recent. Until that point, concrete had been limited to arches and domes, but the invention of reinforced concrete gave concrete forms as much design freedom as steel. However, by incorporating steel, reinforced concrete inherently admitted a factor for deterioration: corrosion (Fig. 3). To this day, we are still battling against this deterioration factor. We do not have the technology to completely stop steel deterioration.

Fig. 1: Pantheon in Rome

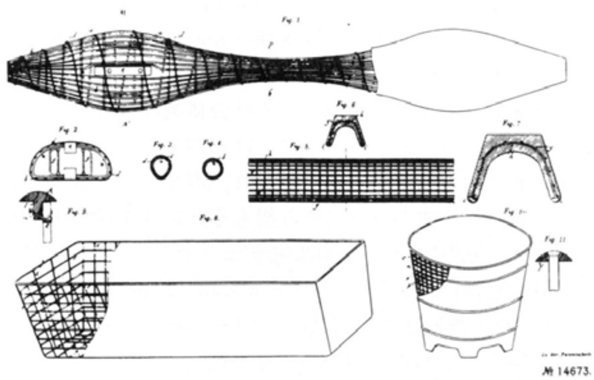

Fig. 2: Joseph Monier's idea

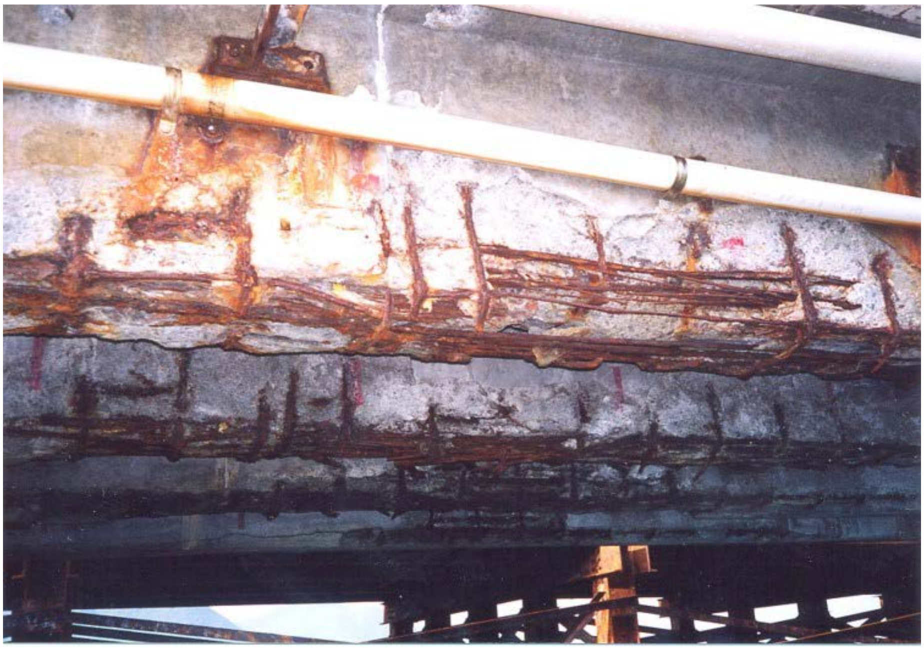

Fig. 3: Deterioration of reinforced concrete

So what can we do to restore the innate durability of concrete? It is already too late to go back to using unreinforced concrete, so replacing steel with new materials that do not corrode is the engineering option with the most potential. Various ideas for structural concrete using carbon fibre and other advanced materials for reinforcement and tendons have been developed around the world. The problem, however, is the high cost. This should be offset by highly durable structures having greatly extended lifetimes that can cut conservation costs significantly. They are therefore superior from the standpoint of life-cycle costs (LCC). However, the ongoing difficulty of determining costs over a lifetime serves as a barrier to their adoption and inevitably leads to decisions based on the initial cost. At *fib*, revisions are currently underway for the Model Code 2020, which will include the adoption of through life management of concrete structures. (Matthew, et al., 2018) If such an approach becomes common in the construction of concrete structures, it will be possible to construct highly durable objects using new materials. This will also contribute to the realization of a sustainable society.

Sumitomo Mitsui Construction (SMCC) has been working on the development of a non-metallic bridge using new materials for reinforcement since 1984. At long last, as of 2019, we are now building a non-metallic expressway bridge. It will of course be the first of its kind in the world. This paper traces our road back to the roots of concrete in Rome.

2. BRIEF HISTORY OF NON-METALLIC BRIDGE DEVELOPMENT

Fig. 4 shows the timeline of our efforts across 29 years, leading to the construction of a non-metallic bridge. The engineering development can be broadly divided into two phases. Phase 1 is SMCC's own engineering development from 1984 to 1990. A 12.5 m span pretensioned beam and a 25.0 m span post-tensioned beam were constructed in our plant. At the time, aramid fibre was used for tendons, and aramid fibre was used in lieu of reinforcing bars for the pretensioned beam reinforcement as well. However, this resulted in a cost that was 2.5 times higher than usual. The cost was too high, so applying the technology to actual structures was determined to be difficult. As a result, development came to a halt at that point.

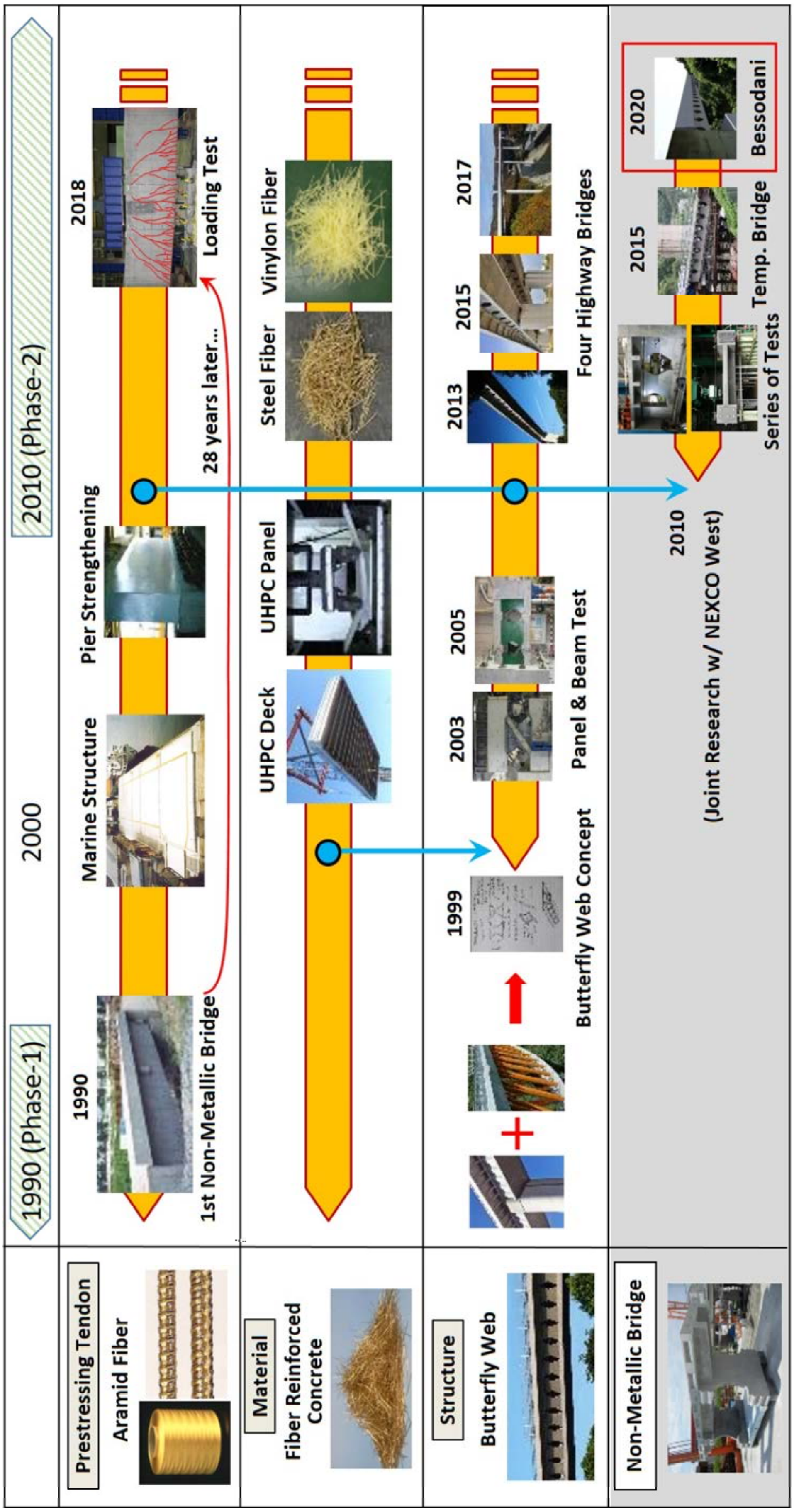

Fig. 4: Timeline of non-metallic bridge development

Phase 2 consists of joint research and development with West Nippon Expressway Company (NEXCO West) that began in 2010. NEXCO West put out an invitation to tender for the development of a non-metallic bridge under the condition that the cost of initial investment is less than 1.5 times the usual cost, and selected SMCC for joint development based on its track record in the field. The major difference between Phase 2 and Phase 1 is that by this time, high-strength fibre reinforced concrete technology had greatly advanced, reaching the point where it can be applied to structural concrete. This technology was a very large factor, making it possible to construct a structure without using aramid fibre for reinforcement, which had been an obstacle in Phase 1. This transformed the process, and eventually led to construction of an actual expressway bridge, commencing in 2019.

As engineers and researchers involved in concrete, we have all been looking forward to the realization of non-metallic bridge technology for a long time, so this is a big step forward. Details of the development in each phase are discussed below.

3. R&D IN SMCC (PHASE-1 FROM 1984 TO 1990)

From early on, SMCC had been looking into the Technora aramid fibre developed by TEIJIN, which, unlike other aramid fibres, is not affected by alkali. This property is an absolute requirement for concrete reinforcement materials. Moreover, reinforcement rods can be fabricated with nodes, similar to deformed bars (Fig. 5). This greatly enhances the rod's bond performance and enables concrete embedment with short development lengths. (Noritake, et al, 1990). After conducting various basic experiments, pretensioned and post-tensioned beams were constructed at the SMCC concrete plant (Fig. 6 and 7) in 1990. Fig. 8 shows the cross-section of each girder. Tendons were distributed in each beam, with 3 x 6 mm diameter tendons in the pretensioned beam and 7 x 6 mm diameter tendons in the post-tensioned beam. The pretensioning method used a temporary anchorage, which anchored by grout bonding inside a 500 mm long steel pipe (Fig. 9). Although this method can only be used once and is costly, it was still applied later on when the use of aramid tendons spread since it anchors securely. Moreover, pretensioned beams were reinforced with axial reinforcement and stirrups made of fabricated aramid rods as an alternative to steel reinforcing bars. However, this turned out to be the main cause of high production costs. Post-tensioned beams had external tendons, which used aramid rods. For anchorage, an anchoring device made of glass fibre was developed (Fig. 10), although development of this anchorage has since stopped due to costs and due to its large size stemming from the use of GFRP.

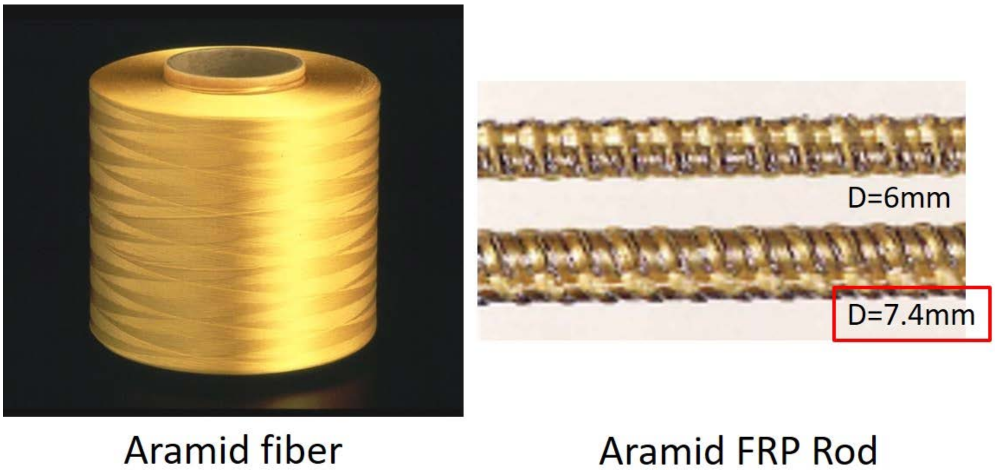

Fig. 5: Aramid FRP rod

Fig. 6: Pretensioned beam

Fig. 7: Post-tensioned beam

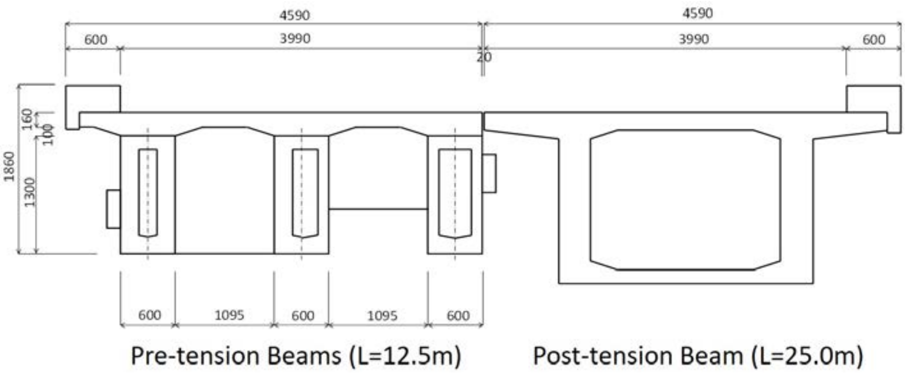

Fig. 8: Cross-section of pretensioned and post-tensioned beams

Fig. 9: Temporary steel pipe anchorage

Fig. 10: GFRP anchorage

Our efforts gained international recognition. In 1996, we received the Charles Pankow Award for Innovation from the Civil Engineering Research Foundation (CERF). However, reducing the costs of this project was difficult, as discussed above, and although we subsequently applied the technology in harbour wharfs and other instances, it did not go further than that and was not used on regular bridges.

The bridges that were built during that time are still in use today. In 2018, one of three pretensioned beams was taken out to conduct loading tests after 28 years of service. (Sanga, et al., 2016) The aramid tendons were in very good condition and showed no signs of deterioration. We also verified that the bending strength of the beam had, naturally, not degraded (Fig. 11). This may well be the first time ever that a new material, which has moreover been used for tendons, was confirmed to be sound after more than a quarter century of use.

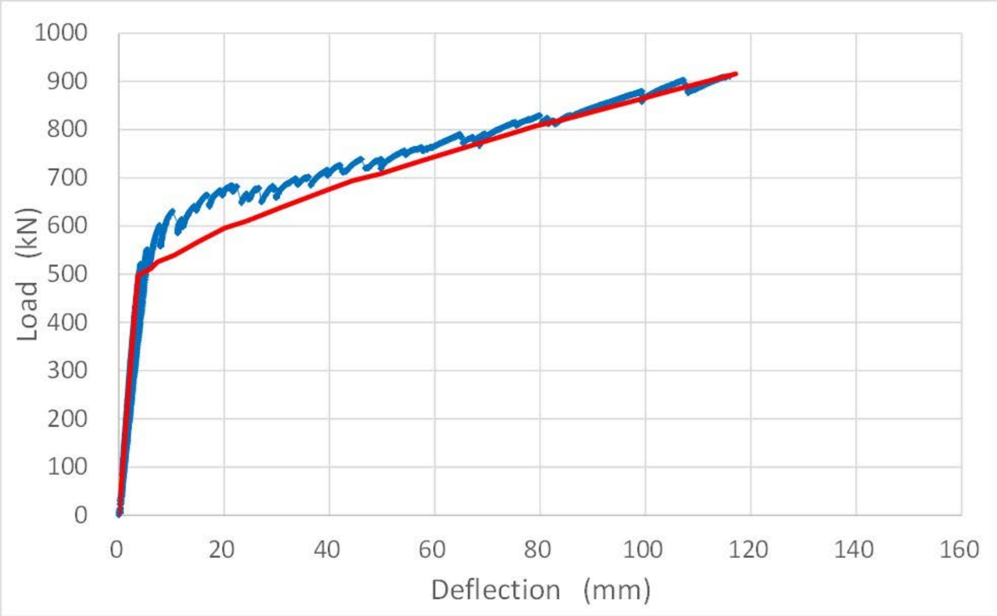

Fig. 11: Relationship between load and deflection

4. JOINT RESEARCH WITH NEXCO WEST (PHASE-2 FROM 2010 TO PRESENT)

In 2010, NEXCO West invited tenders for joint research on a non-metallic bridge. The terms for the non-metallic bridge required that the initial cost be kept within 1.5 times that of the typical bridge. It had been impossible to meet this requirement with Phase 1 technology. This time, however, we determined that it would be possible if we used high-strength fibre reinforced concrete, a technology which was ready to be implemented, for the concrete and omit the use of new material reinforcement. Recognizing SMCC's track record in the field, NEXCO West started the joint research and development project with SMCC.

The R&D collaboration began with the concrete mix. To minimize costs, we set the target compressive strength of concrete to 80–100 MPa. Ultra-high performance concrete (UHPC) already had a track record at the time, but we did not utilize it due to its cost and because, as a mortar, the shear strength is not as high as the compressive strength. The mixing proportions that we decided on are shown in Tab. 1. For the fibre reinforcement, we obtained data on steel fibres and vinylon fibres (Fig. 12), and decided to use the most appropriate fibres for each location. For the web and lower deck slab, 0.2 mm diameter high-strength steel wires that are 22 mm long were used at a volume fraction of 0.5%, since this had the highest shear strength. NEXCO West consented to the use of steel fibres, as it had been shown that even if they rust, the rust would be confined to the concrete surface. In addition, we decided to use concrete reinforced with vinylon fibre for the upper deck slab and concrete barrier, where the deterioration requirements are stringent because of the use of de-icing salt. For the upper deck slab in particular, we verified that the slab meets the required performance for expressways through a fatigue running test shown in Fig. 13 and 14.

Tab.1: Mixing proportions of concrete

Name	Steel Fiber		Slump	W/B	Content (kg/m^3)				
	sort	volume	(cm)	(%)	W	C	SF	S	G
SW	SW	0.50%	20±2.0cm	25	175	630	70	408	596

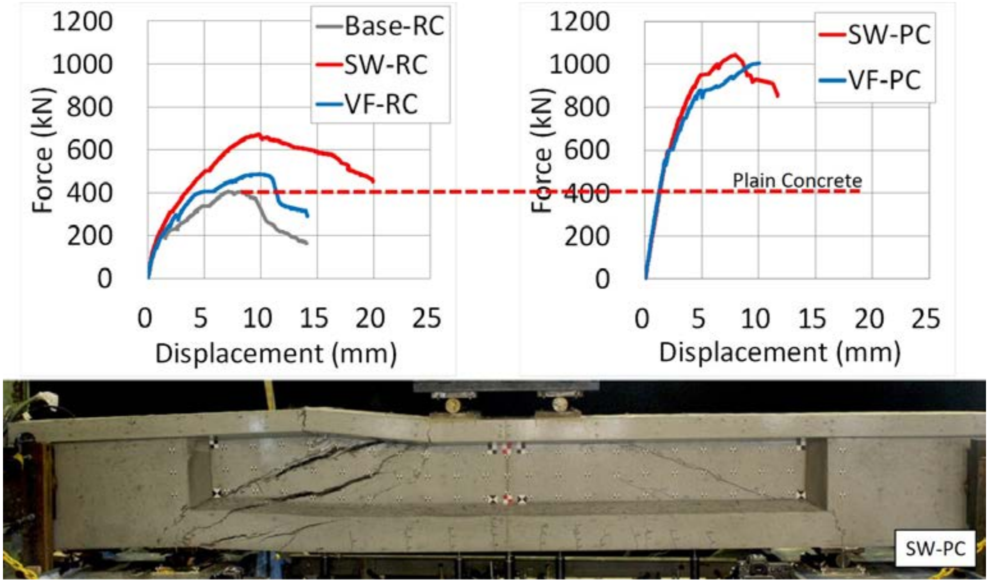

Fig. 12: Relationships between force and displacement

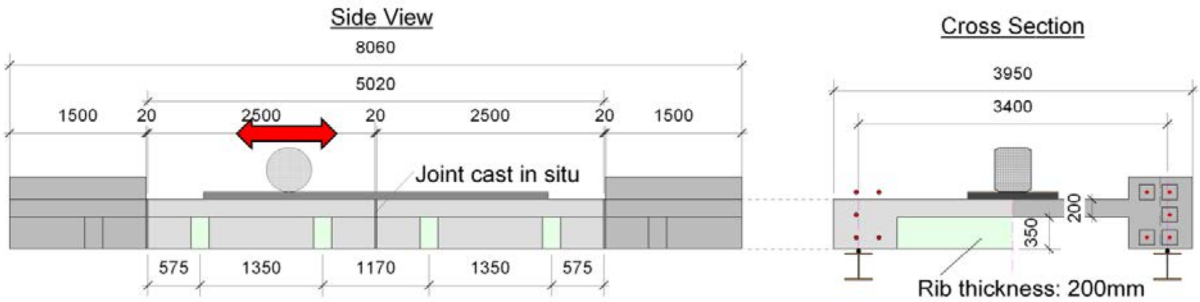

Fig. 13: Outlines of fatigue running test


Fig. 14: Fatigue running test

Next, the structure was determined. The formulation of high-strength fibre reinforced concrete mix that had been decided was still more costly than normal concrete. We therefore decided to use butterfly webs (Kasuga, 2016) developed by SMCC (Fig. 15) for the girders, in order to minimize the amount of materials. The butterfly web structure was like that of a double Warren truss, but executed with concrete panels. This resulted in a significant reduction of materials, as 30% of the typical web had been turned into openings and we were able to reduce member thickness of webs to 15cm. A load bearing test for the structure was carried out on a half-sized girder (Fig. 16), which showed that the behaviour of the new structure can be modelled by nonlinear analysis (Fig. 17). (Ogata, et al., 2016) For the non-metallic bridge, the butterfly webs were fabricated first, after which the upper and lower slabs were added to assemble a box girder (Fig. 18). The structures of the joints between the web and slabs were also designed and capacity against bending in the transverse direction was verified (Fig. 19).

Fig. 15: Butterfly web bridge

Fig. 16: Load bearing test

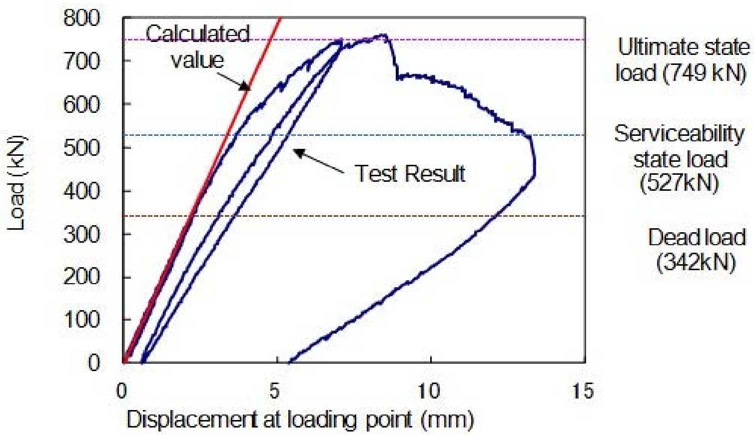

Fig. 17: Load vs displacement relationship

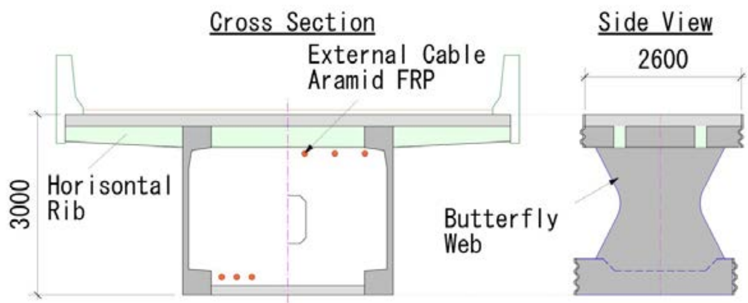

Fig. 18: Outlines of assembling non-metallic bridge

Once the materials and structure were fixed, and data for the design collected, the next step was to apply the technology to an actual structure to obtain practical data. This was first done on a temporary bridge that was to be used as a construction access road for two years

(Fig. 20). The temporary bridge with a span length of 14 m and a width of 6 m was constructed using the exact same process as an actual highway bridge, in order to obtain data over the two-year period. Eight precast segments were produced at SMCC's concrete plant (Fig. 21), transported to the site, and erected over falsework with a crane. After completion, constant bridge monitoring and vibration tests using cranes were conducted to make sure that there were no behavioural problems throughout the two years.

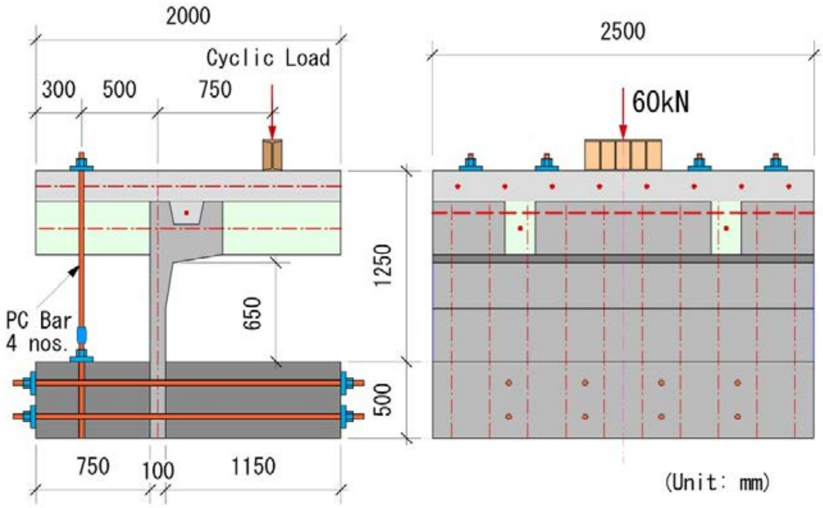

Fig. 19: Outlines of joints between the web and slabs

Fig. 20: Temporary non-metallic bridge

Fig. 21: Precast segments

Finally, the technology is being applied to a highway bridge, which is our ultimate goal. The Bessodani Bridge is a 25 m long simply supported girder bridge (Fig. 22 and 23) for the Second Phase of the Tokushima Expressway in Tokushima Prefecture. Segments are currently under production and construction will begin by early 2020. The bridge specifications are 100% non-metallic. Segments will be erected on an erection girder, and the concrete barrier and upper slab are made of fibre reinforced concrete using vinylon fibre. Moreover, the drainage systems are made of FRP to extend their service life as much as possible. Cost cutting measures were further implemented for this project. In Phase 1, aramid fibre tensioning was performed with single-use grouted temporary steel anchorages. This time, however, a reinforced plastic wedge jointly developed by SMCC and Sumitomo Bakelite is used (Fig. 24). Although the wedge sleeve is made of steel, it is economical as it can be used 10 times. We are also currently in the development stage of a plastic wedge for long lasting use.

Fig. 22: Bessodani bridge

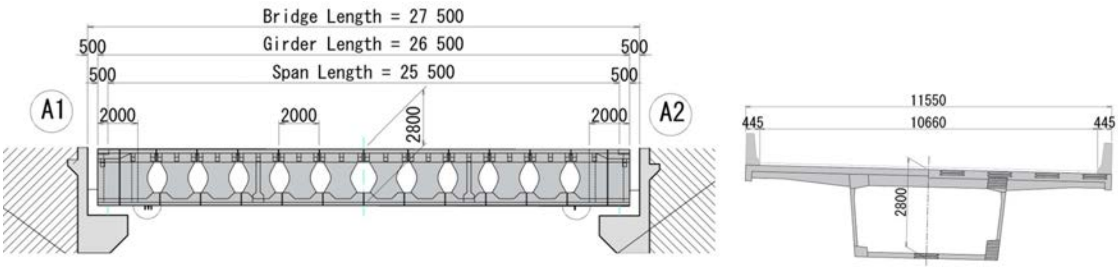

Fig. 23: General view of Bessodani bridge

Fig. 24: Reinforced plastic wedge

5. CONCLUSIONS

Moving from the development to the implementation of a non-metallic bridge took about 30 years, going through various phases. The largest factor in that period is the engineering innovation of fibre reinforced concrete. Putting this technology into practical use, we were able to immediately resolve the problem of cost, which had been an impediment in the early stages. In Japan, life-cycle costs for expressways are generally estimated at 2 to 3 times the initial cost. Therefore, spending 1.5 times the usual initial investment on a highly durable bridge is deemed quite reasonable. Going forward, we also have to study how to reduce the factor of 1.5 in initial investment by considering which of the non-metallic materials is most appropriate for each type of structural member. In addition, we believe that a ranking of the extent to which the structure is non-metallic should be used to optimize designs.

Through the short 150 years of history of reinforced concrete, as well as the 80 years or so of history of prestressed concrete, concrete has come a long way along the road that leads to Rome, back to concrete with an innate lifetime of 2000 years, taking us to an age when the long-cherished desire of researchers and engineers to be liberated from the chains of deterioration will be achieved.

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FROM RESEARCH TO INNOVATION: CASE STUDY OF A ROAD PAVEMENT RESEARCH PROJECT

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SUMMARY

The major job of research is to create such intellectual asset which has benefits on the socio-economic level. This goal can be realized only if researches are application oriented, tightly together with development, even in case of basic research.

This paper introduces an application-oriented research project, which originated from a controversial research idea. The final project includes all the elements of the innovation chain: basic research, applied research and innovative development.

The project deals with pavement design. The basic idea was a new approach for modelling the characteristics of asphalt mixtures. However, this approach was incompatible with the current paradigm, so other research and development activities were needed to exploit the benefits of the results of the basic idea. For this purpose the project was extended to include a finite element pavement model, a pavement diagnostic system, and some software development. All along, the focus was on the applicability of the results.

1. INTRODUCTION

Research is only useful for the society if it addresses real needs and creates benefits in the near future. From the point of view of profit-oriented organizations, this requirement is stricter, as these organization demands well-defined and tangible economical benefits. An idea, a discovery, however brilliant it is, in itself is not enough, unless a long queue of finance-thirsty innovations follows it. Without that the social benefit of a discovery is somewhat minimal.

The mission of research is to produce results with benefits on the social and economical level. So every research should focus on providing benefits. And benefits rise only if the results of the research can be applied to the more efficient execution of a certain task. Thus, research must orient towards practical application. The process, which makes the exploit of research results possible, is called the innovation chain, and consists of a chain of research and development activities.

According to the linear model, the innovation chain can be divided into three main parts: basic research, applied research and development. Or using another terminology: discovery, invention, innovation. This model assumes that the phases follow each other sequentially as insulated activities. And this is a weakness, because it gives a too big importance to research, overshadowing the other phases. But without invention and innovation, the results never make to the market.

These elements of the research and development process are related and distinct as well. Discovery creates the theoretical background on which the other elements can rely. Invention is the creative step, which creates a potentially useful something based on the existing knowledge, as an applied research based on the results of earlier research activities. Finally, innovation, consisting of numerous small practical improvements, makes it possible to utilize in practice the results of the innovation process.

One of the problems the research activity should handle is to overcome this linear perspective. The three elements of the innovation chain should be considered related, and even at basic research activities the potential application should be kept in view, together with the necessary inventions and innovations needed to this potential application.

Innovation connects directly to application. Here, there is no problem with application-oriented approach and benefits. The innovation either produces profit or other tangible benefits, or not. In the first case the innovation is desired, in the latter it is not and unnecessary.

Going up in the chain, the situation becomes a little more foggy. The invention phase is still close to the practical application to make assessing its benefits and applicability possible. However, the results can not be exploited directly, it requires following innovation. Applied research or invention usually looks for a theoretical solution of certain practical problems, but without further development these theoretical results will not be available to exploit.

Basic research is very far from the final practical application and benefits. Long decades might pass before the potential applicability of the results of a basic research becomes apparent. And then comes only the rest of the innovation process. Astrology and quantum mechanics might be mentioned here as an example. So, it is usually very difficult to see the practical benefits of basic research at the beginning. But we should continuously strive for identifying them, however theoretical or distant they might be. So, even if we are in the field of basic research, we should consider the other elements and the potential further research and development activities needed for the practical applicability of the results, however distant they might be.

This paper aims to demonstrate this kind of application-oriented thinking in relation to a current research project.

2. THE INVENTION LOGIC: FROM BASIC RESEARCH IDEA TO PRACTICAL APPLICATION

Research usually starts with an interesting question or problem. There is an idea to solve the problem. But the questions are: are these results important? For what and how can they be applied in practice? How can the whole innovation chain constructed? Our case study gives an example how these questions can be answered.

In the field of technological design of road pavement structures, science reached an important milestone. The rapid development of computing capacity enables the application of more complicated models in practice, such as models based on the mechanics of materials instead of semi-experimental models. Overcoming certain challenges, impossible in the past, became achievable in the present.

Exploiting the possibilities of the digital economy has a very significant importance in the field of road pavement design. Given the value of the road infrastructure and the costs of road constructions, every little technological decision has heavy financial consequences. Every self-conscious country came to the decision that more advanced road pavement design methods are required.

A design methodology is considered adequate, if

- it is based on the strength of materials and mechanical analysis;
- gives clear-cut results;
- the methods of new pavement design and of reconstructional design are consistent.

In the majority of cases, road pavements are made of asphalt courses. Asphalts are well researched in this field, as a very important element of road structures. However, current models has certain areas where theory can not answer the behaviour of asphalts properly.

And here comes the radically novel idea, from which the whole research grew out. Dr. György Gajári assumed a new approach, and instead of the current paradigm he applied a hypoplastic model to describe the behaviour of asphalts. This approach considers asphalts as barotrop stone structures, pre-stressed by bitumen. This model is completely based on the strength and the mechanics of the materials, and can experimentally be validated.

According to Dr. Gajári's results, the hypoplastic model of asphalts is more appropriate to describe the behaviour of asphalt mixtures than the current ones. Also, this hypoplastic model can explain those phenomena which are problematic today. However, this approach changes the asphalt mixture design principles drastically, the results imply the benefits of the application of softer bitumen in mixtures, which, of course, opposes the current mixture design principles.

So, although this hypoplastic approach is very interesting, and is worth to be explored in depth, its practical applicability is approximately zero, because it can not be integrated into the current paradigm. Simply, the basic principles differ. This is a research, which is interesting, but leads nowhere. Off with it.

But, if we try to tackle the problem of applicability of this hypoplastic approach, interesting things can emerge. The first question: how can this approach be integrated to the pavement design methods? The answer is easy: a new design methodology should be developed, which can handle both the methods based on the current principles and the ones based on the hypoplastic approach. So, if we want to research the hypoplastic model of asphalts, a new design method will be needed as well.

However, the development of a design methodology requires significant financial sources and time. This is clearly out of the possibilities, and not really necessary. A simpler solution is the adaptation of an existing methodology. Currently there are two major approaches in the field, the French and the German one. Based on its features, the German methodology was more suitable to our purposes, so we chose that one.

As we have seen it earlier, there is a great need to create more advanced pavement design methods than the current ones. We are in the invention phase now: a theoretical solution is needed for a practical problem. The potential exploit of the basic research idea made it necessary to develop better pavement models and design principles.

Still, these results can not be applied directly in practice. Finite element models are complex, and for the everyday tasks are too complex. And complex models just offer too many possibilities for error in exchange for slightly better results in the majority of the cases. In the current practice, different pavement structure configurations are defined in advance, among which the designer can choose. So, something should be done to improve the practical applicability of the results.

This is the innovation phase now. In this case, practical improvements meant software development. using the open-access modelling concept, we separate the modelling task from technology design tasks, and create a simple tool for the practicing designers to make mechanical analysis using finite element model of road pavements.

Going through this chain of logic, looking for the potential application of the results, we have created a series of research and development tasks. The innovation chain has the following element in our case:

1. Basic research: hypoplastic asphalt model
2. Applied research: pavement design method based on a finite element model
3. Innovation: software development, which makes it possible to use the earlier results in practice.

In our case we were lucky, because we could integrate the whole innovation chain in one project. But even in those cases, where this is not possible and these elements widely separates in time, potential inventions and innovations should be considered for the sake of a future practical applicability of the results. Research and development should advance tightly together.

3. THE PROJECT: DEVELOPMENT OF MECHANICAL MODELS OF ASPHALT MIXTURES AND PAVEMENT STRUCTURES

The goal of the project is to develop a mechanical model of asphalt mixtures and the pavement structures, which is:

- up-to-date and analytical;
- describes the requirements for the pavement structure by performance-based measures;
- considers the mechanical parameters of the applied materials;
- provides a mutual theoretical background for the design of new pavements and pavement reinforcements;
- able to handle new materials and technologies, and the innovative methods of increase of pavement life span in an exact way.

3.1. Asphalt model

Some experiments imply that the stiffness of asphalts is not constant, but pressure-dependent. The hypoplastical approach, which considers bitumen as a liquid component, can describe this phenomenon, explaining this pressure-dependency with the pre-stressing effect of the cooling and shrinking bitumen.

In the research we have formulated this hypoplastic asphalt model, and its finite element model is implemented.

3.2. Pavement structure model

The project adapts the German methodology for pavement structure model. This model is implemented by a linear, two-dimensional finite element formulation. The original model is extended by taking into account additional factors, such as climate, asphalt temperature, soil characteristics, etc. This extension makes the model much more complicated than the original one, and requires much more data and computing.

To provide the necessary data, we create additional models to the pavement model. By the use of these models, necessary data can be created for the model. The designer should provide only a few input data, mainly about the location of the road section. These additional models include a meteorological model, soil model, and a traffic forecast model, on a national level.

As the hypoplastic model is in research phase, and confronts the current paradigm of the field, we have included into the pavement model both the traditional and the hypoplastic asphalt model. Theoretically the user can choose which model he wants to use, but in default the traditional model is offered, and the hypoplastic method is currently reserved for research purposes.

3.3. Pavement diagnostics

The increased number of parameters in the pavement structure model requires increased number of input data. Some of them, such as meteorological data, are independent from the pavement, and can be collected otherwise. However, there are certain pavement characteristics, which is needed for the new model, but the current diagnostic procedures can not provide it, or can not provide it efficiently. Such data are for example the water content of the pavement and the earthwork.

To overcome this problem, an efficient pavement diagnostic methodology is needed. This procedure should be able to determine reliably the state of the pavement course in a cost and time efficient way with marginal disturbances for the ongoing traffic, both for quality assurance and technological design purposes. In the project we develop a georadar based diagnostic system to measure the necessary data. Calibrating the measures with CT analysis and laboratory tests, a material model, suitable to describe the characteristics of pavement courses and the earthwork, can be developed. Besides these, the measures can be calibrated with geophysical and laboratory tests to reveal flowing water in the pavement and the earthwork.

This diagnostic methodology will make it possible to continuously monitor the road network and the deterioration of the roads. This is a sideline of our research, but solves a very important problem in road asset management. And it also helps the continuous evaluation and improvement of our model.

3.4. Design software

The software implements the open-access modelling concept. This concept separates the modelling process into two distinct modules: model-building and carrying out forecasts

(Fig. 1). The basic approach behind the concept is that the most demanding tasks of model-building, implementation, calibration and validation, data collection and model update, which create the strongest barriers in practice, should be removed from among the everyday tasks.

Pavement design is built upon the model as an open-access service. Designers need not be concerned about modelling tasks, as they only have to define the modelling environment of the pavement. They estimate the expected value of the input data of the models and define the considered scenarios.

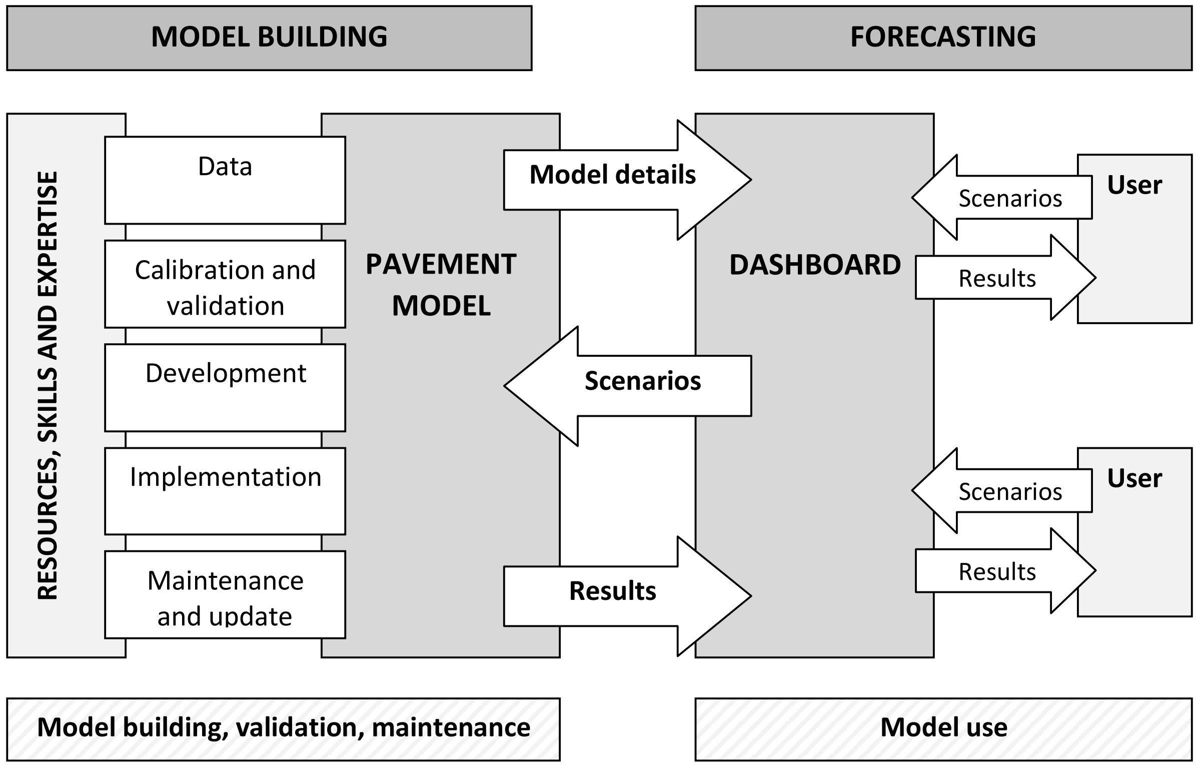

Fig. 1: The open-access modelling concept

This concept gives more value to the users than the usual modelling tools and concepts, because above the modelling implementation it contains the majority of input data as well (e. g. meteorological data, traffic forecast, road network etc.), thus, users do not have to collect, analyze and code them. Beside the built-in data the concept gives significant flexibility to the users, because any parameter can be changed. However, users do not have to parameterize the whole model with the already known static data, but they can focus on the set of parameters which are important for the analysis of the given task.

For the implementation of this open-access modelling concept a modular framework is developed (Fig. 2). The basic concept of the implementation is a client-server relation. The modelling core and the database run on a server connected to the internet, which can be accessed by the users through the clients.

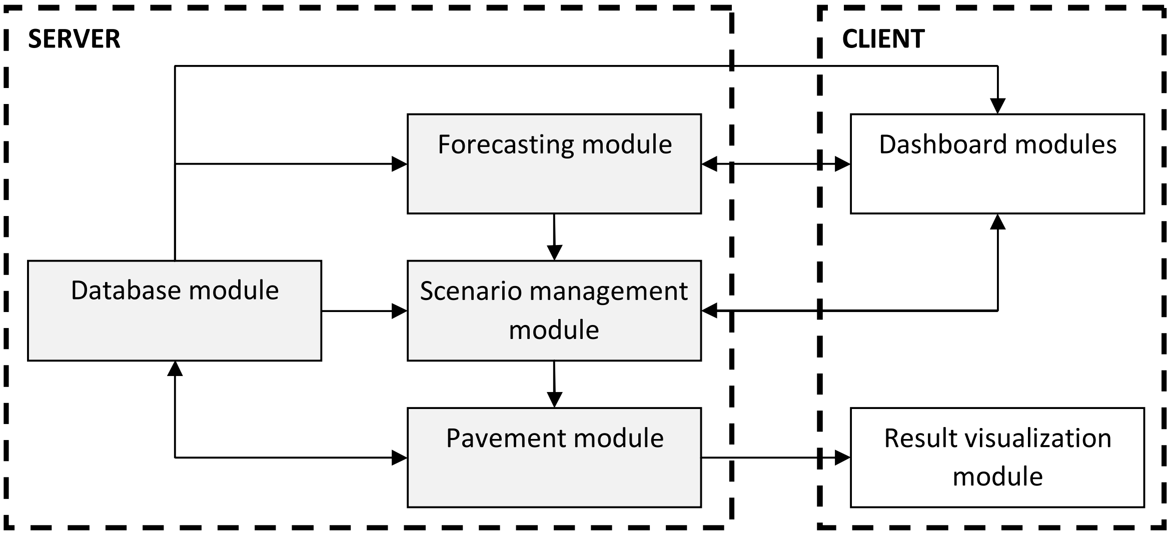

Fig. 2: The modules of the modelling framework

The dashboards provide full access to the policy variables and the variables of the transportation system. Every dashboard has three different levels, according to the expertise or the intentions of the users. The more detailed dashboards include the simpler ones, providing all their functionality for the high level users as well.

The first level is for basic level users, who do not require wider customizing options or who do not have the skills to directly predict the future values of the input variables. For them the dashboard offers the choice among the most realistic forecast and its very optimistic, optimistic, pessimistic and very pessimistic versions. With these scenarios users can model all the probable environments of the future to predict the values of the input variables.

For the advanced level users the second level offers more possibilities: besides the basic scenarios they can manipulate major modelling parameters. This provides more flexibility but means more liability as well, as the possibility of making errors increases. The third level is for the expert level users, who can directly define the value of any input variable.

4. CONCLUSIONS

In digital economy innovation has become the most important factor of the economic development, innovations are created in a rapidly accelerating rate. Today innovation is a primary element of the society, for which cutthroat international competition goes on. Research cannot be made for its own sake, it should relate to social or economic needs. Even the most brilliant discovery is worth nothing, if it stays in the drawers and does not get to practical application. And this requires invention and innovation. Research and development should progress tightly together.

In this case study we showed how this innovation logic can be applied for a research idea, to create a practically useful innovation. The starting point was a very interesting new idea, which was incompatible with the current scientific paradigm. So, it had very little practical value. Applying the innovation logic we identified the necessary inventions and innovations which might make it possible to use the results of the basic idea in practice. During this analysis we found areas in the area of pavement structures, which needed improvement. And with the suitable improvements, the basic idea became compatible with the system, and the innovative software development removed the barriers of everyday application.

MODERN HIGHSPEED TURNOUT SYSTEM SOLUTION - FROM GEOMETRIC AND STRUCTURAL REQUIREMENTS TO SIGNALING INTEGRATION

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SUMMARY

High-Speed turnouts are highly complex subsystems of any railway infrastructure and ensure the flexibility of train operation as they merge and connect open railway tracks. Meeting highest safety requirements, turnouts themselves consist of a large number of different components. Based on the knowledge of the interaction between the vehicle and the track in a turnout, design improvements can be implemented for the many individual components of the turnout. The wheel rail forces, the impact loads and the strains on infrastructure and vehicles are significantly reduced by innovative measures and design changes. The geometry of the turnout can be significantly adapted to support the vehicle run and moveable frogs can be implemented, both resulting in significant improvements of the whole system. A well-balanced elastic support of the sleepers in a turnout as well as the use of materials featuring high resistance against degradation as well as highest profile stability also significantly contribute to a good long-term behaviour of turnouts in the track. Well considered switching and locking system in combination with monitoring and prediction of certain arising problems are explicitly important for safe operation and providing highest availability results.

1. INTRODUCTION

The continuous efforts in improving design and materials of railway components has not only preserved the performance and economic standards of railway infrastructure, it has also lifted these parameters to a higher level. Remarkably, these upgrades have been achieved, despite the steadily increasing demands that have been observed, over the past decades. Nowadays, speeds up to 350 km/h are standard in high-speed networks. An end of this railway boom and as a result a stagnation of the increasing loads cannot be foreseen. On the one hand, this continuously increasing traffic in the railway sector leads to a healthier environment. On the other hand, high train frequencies, higher loads and increased speeds contribute to an accelerated deterioration of the infrastructure components and leads to increased maintenance requirements.

Therefore, further innovation in the railway industry is essential today. A modern interpretation of innovation involves much more than isolated efforts in improving isolated components like the rolling stock or infrastructure. However, the improvement of single components can still be useful in terms of maintenance and service life of the respective product bringing an upgrade to the overall optimum. The fully potential of an overall optimum can only be exploited by considering the overall system and especially the interaction between vehicle and track or wheel and rail, respectively. For such a system optimization, the individual components should not be receivers of stresses like pressures, slip forces, impacts, etc., only. The active role of these components in influencing the forces acting on the vehicles and the infrastructure must be considered in modern railway infrastructure solutions. Therefore, this approach applies to both, the infrastructure and the rolling stock and includes open track sections as well as turnouts.

In modern High-Speed networks, turnouts are one of the most important infrastructure sub-systems. It has always to be taken into consideration that due to the general tasks and the geometric design, each turnout itself represents a homogeneity interruption in the continuous track by nature.
Turnout construction is faced with the following challenges:

- High train frequencies and speed
- Limited possibilities for maintenance – mostly at night
- Highest focus on safety requirements
- Most challenging exposure for the railways superstructure

These challenges have to be met by the following improvements:

- Robust turnout technology
- Easy and simple maintenance procedure
- Long maintenance intervals
- Highest safety
- Predictive maintenance requirements

This article will give an overview of turnouts as a system component as well as an individual infrastructure subsystem by itself. Specific actions to decrease the forces for both, the rolling stock and the infrastructure, result in a higher system optimum and are highlighted in this article. An optimized turnout, with geometrical optimization of the switch and structural improvement of the crossings, significantly reduces the lateral and vertical forces. Thus, the impact forces in the crossing area acting on vehicles and infrastructure can be largely avoided. Additionally, by using softer bearings at the turnouts, the ballast bed will be protected but also the dynamic peak forces in the wheel/rail contact can be reduced.

Two requirements have to be fulfilled to guarantee the long-term success of these actions. On the material side, the ideal geometry over the whole turnout has to be preserved as long as possible to guarantee the best possible contact conditions for the vehicles permanently. Such an improvement on the material side can be realized by using rail steels with highest resistance against wear and rolling contact fatigue for highest profile stability. The second requirement is homogeneity. This does not mean homogeneity of single components but the homogeneity of the vehicle and infrastructure interaction. To reach an optimum in the homogeneity, the distribution of elasticities has to change with regard to the position in the switch area, guided by the boundary conditions of the system. The geometry in the transfer area must be adapted to the effects caused by the transfer points based on the running behaviour of the vehicles.

2. DYNAMIC OPTIMIZATION OF TURNOUT GEOMETRY AND SWITCH

The turnout geometry and the switch design is decisive for the management of the forces coming from the rolling stock. High-speed turnouts allowing speeds of up to 350 km/h in the main route and up to 220 km/h in the deviation. The speed in combination with uncompensated lateral accelerations and disturbances is contributing with the 2nd power to the loading of the vehicle and the infrastructure. For that reason, it is enormously important to replace simple circular turnouts by turnouts with transition curves (cosinuide, clothoide curves) to avoid abrupt change of uncompensated accelerations when the train is riding over (Megyeri, 1985), (Holzinger, Fritz 1990). If this is not taken into account, high interface forces introduce severe damage to the switch components with a significant reduction of the riding comfort. The consequences would be high maintenance requirements in combination with a reduced service life.

Fig. 1 and Fig. 2 are providing a comparison in uncompensated lateral accelerations between a parabolic turnout geometry and a clothoide turnout geometry. Especially at the entry of the turnout in a crossover arrangement, the clothoide geometry shows significant advantages against the parabolic geometry. This situation even gets worse with the use of circular turnouts.

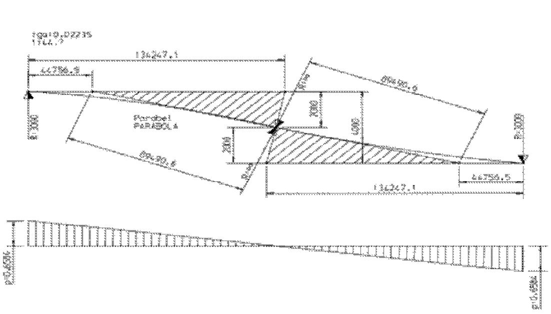


Fig. 1: Uncompensated accelerations in a Parabolic Turnout for 160 km/h

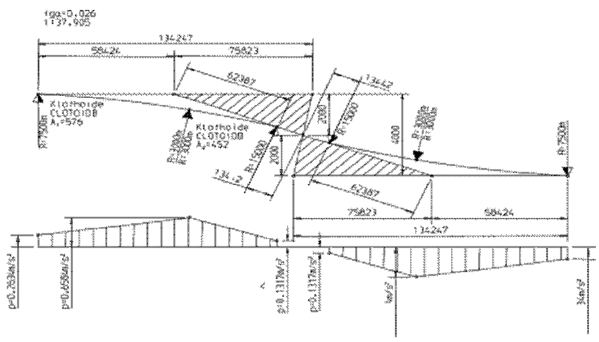


Fig. 2: Uncompensated accelerations in a Clothoide Turnout for 160 km/h

State of the art in this field is the calculation with Multi Body Simulation tools like NUCARS or SIMPAC, which are producing reliable and realistic results in the sense of forces (vertically, laterally) and accelerations when a vehicle is running over a turnout (Fig. 3), (Klauser 1995).

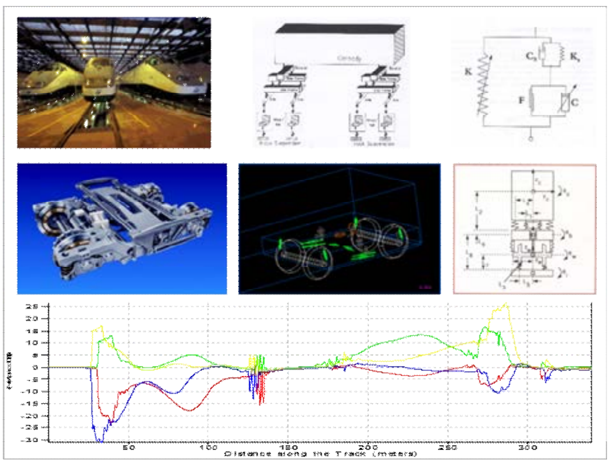


Fig. 3: Configuration for Multy Body Simulation in NUCARS Code

Further potentials can be derived from the geometrical design of the switch. In a plane line the rails are providing a continuous running surface for the wheels, whereas in turnouts the contact conditions for the wheels are changing in the switch as well as in the area of the crossing quite significantly. In the open track, the vehicle is steered and a stable vehicle run is ensured by the profile functions of the wheel/rail contact geometry. Thus, the running behaviour is heavily dependent on the contact conditions between wheel and rail. A higher speed requires a better running stability and this can very basically be described by a longer wavelength of the sinus function of a freely moving wheelset. In a conventional tangential turnout this smooth behaviour is interrupted. Because the wheels have to run from the stockrail to the switchblade the self-steering effect is no more there and the axles are tending to turn against the switchblade. They are starting to act with a higher angle of attack like a grinding disc resulting in increased wear, plastic deformation and breakouts. For that reason, a special switch design was developed to compensate these negative effects. It is known under the name KGO (Kinematic Gauge Optimisation). It leads to a significant reduction of the occurring forces and an extension of the component service life, combined with an increased running comfort (Ossberger H., 2005), (Ziethen et al, 1990).

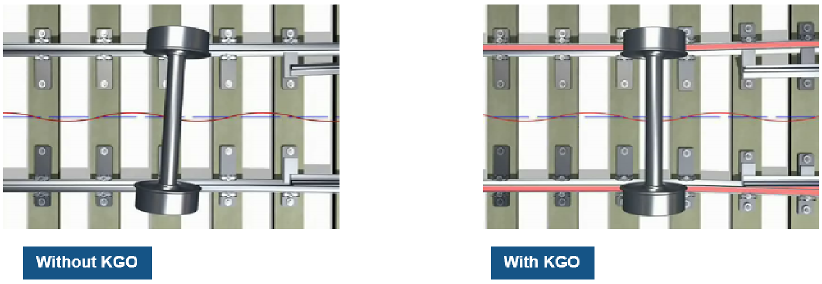


Fig. 4: Comparison vehicle movement in a turnout without and with KGO

When a bogie/vehicle is entering a conventional turnout, the normal running behaviour is interrupted. To prevent the first axle to turn against the switch blade, the KGO design compensates deficits with less rolling radii differences (Fig.4). This is achieved with curving the stockrail to the outside and to allow the two wheels running on similar diameters (Fig. 5).

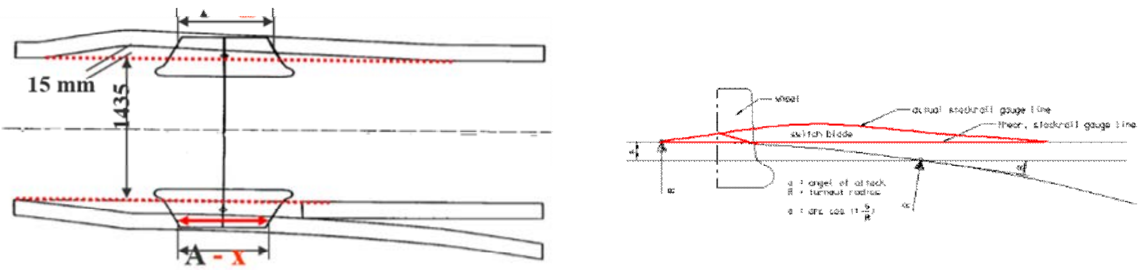


Fig. 5: Design principles of a KGO Switch

The significant reduction of the rolling radii differences is minimising the negative turning effect of the first axle despite a complex transition geometry in the switch. Multi Body Simulations and measurements in track are confirming the positive influence of such a gauge widening in the range of 10 to 13mm in order to get rid of this negative turning effect. With such a design, the lateral forces can be reduced by up to 30% compared to conventional turnouts (Fig. 6), (Ossberger H., 2005).

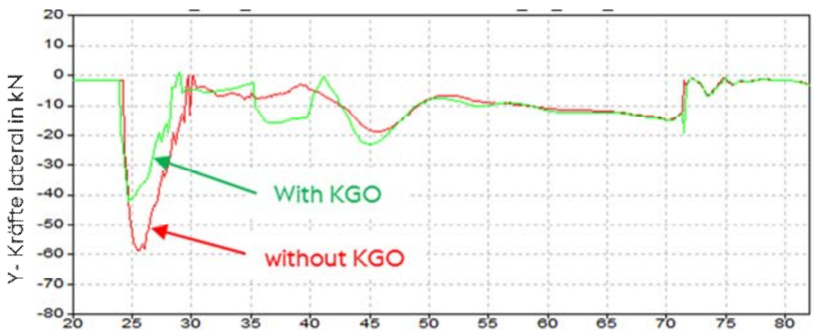


Fig. 6: Comparison of lateral forces in 1:20 turnouts without and with KGO

Beside the positive influence of the running behaviour such a KGO design has a second big advantage. The wear reserve of the switchblade increases significantly. In the critical area of the switchblade the thickness of the blade becomes double, what has in fact a very positive influence to the lifespan of the blade (Fig. 5).

The use of clothoide geometry turnouts in combination with KGO has an enormous influence on the improvement of the total system. The interface forces between wheel and rail are reduced and this contributes to the stability of the whole turnout panel. Initially, this design has made the brake through in High-Speed but meanwhile it is used in all areas like mixed traffic, heavy haul and for metros quite successfully. Fig. 7 and Fig. 8 are showing two examples of Clothoide Tunrnouts for High-Speed.

Fig. 7: Clothoide High-Speed Turnout with KGO on slab track in South Korea

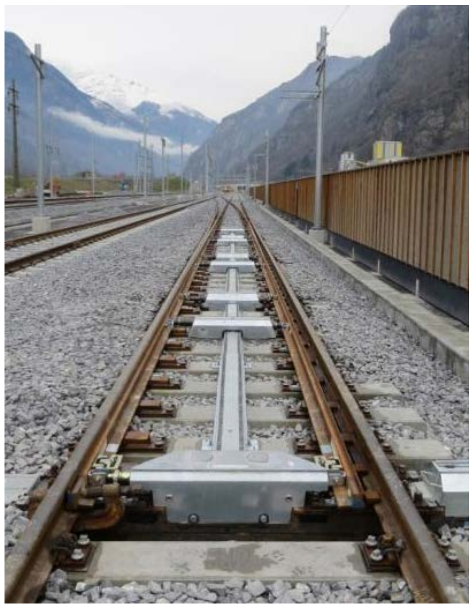
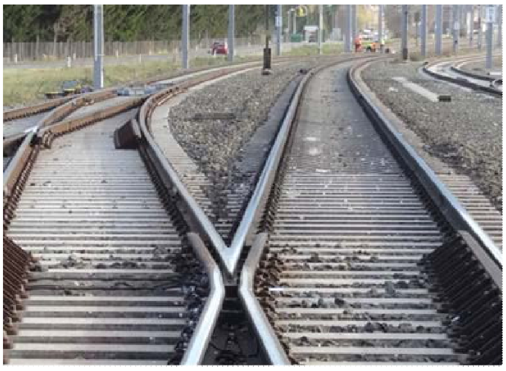


Fig. 8: Clothoide High Speed Turnout with KGO on ballasted track in Austria

3. DYNAMIC OPTIMIZATION OF CROSSINGS

Beside the switch, the crossing is the 2nd critical part in a turnout. The crossing is the area of a turnout where the mainline gaugeline is crossing with the gaugeline of the deviating track. It has to accommodate the wheels with the flanges. This is achieved with a fixed crossing with open flangeways (Fig. 9) or with a moveable point crossing where the crossing point is moved to provide an open flangeway and a continuous running surface for the wheels (Fig. 10).



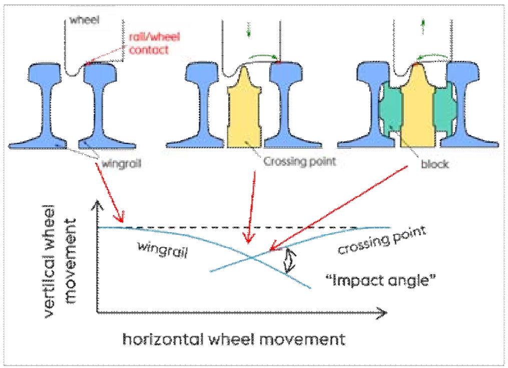


Fig. 9: Fixed Crossing and wheel movements

A fixed crossing, which is state of the art in many traffic areas, is no more the best choice from the point of safety and maintainability. It introduces high dynamic forces to the system.

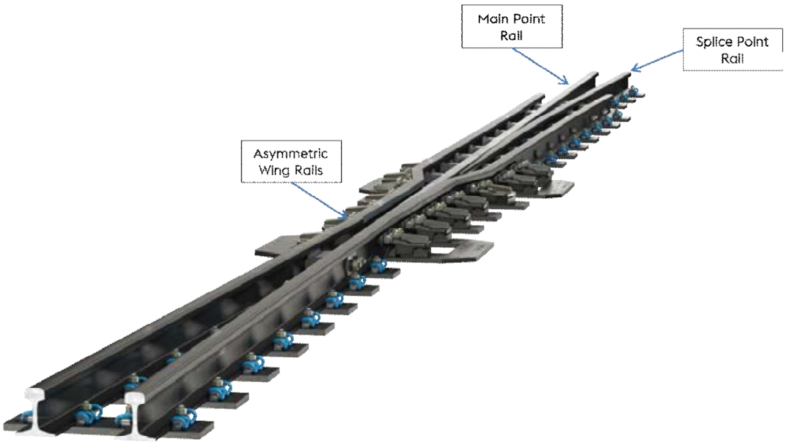


Fig. 10: Moveable Point Crossing

From the point of safety, a moveable point crossing is compulsory according TSI rule for speeds beyond 250 km/h. From the point of maintenance requirements and lifespan, we recommend the use of a moveable point crossing already for speeds higher than 200 km/h. Especially for high speed it has an extraordinary importance to provide a continuous and smooth running surface for the wheel to keep the dynamic impacts and forces to a minimum. However, the positive effect of moveable point crossings should be considered and examined for other types of railway operations as well.

Fixed crossings have the disadvantage of a reduced bearing surfaced due to the flangeways. The transition from crossing point to wingrail causes the wheel to move downwards and upwards resulting again in high impacts (Fig. 9). The reason for this kinematic special situation lies in the conical shape of the wheels in combination with a range of different wear stages of the wheels.

The most efficient way to improve the transition in the crossing and to meet the safety requirements for High-Speed is the use of a moveable point crossings. Contrary to fixed crossings, there is no interruption of the running surface. The crossing point is moved from one wingrail to the other wingrail to provide the necessary flangeway on one hand and a continuous wheel-running surface on the other hand (Fig. 10), (Ossberger U. 2015).

On top, there are advantages from the point of overrunning kinematics in a moveable point crossing which have a positive effect to the frictional work and consequently a positive effect to the RCF damage mechanism for the crossing as well as for the wheels.

And last but not least, moveable point crossings are providing advantages against airborne and structural born noise, which becomes increasingly important for density populated areas. Contrary to a fixed crossing, there is additional DLD (drive, locking and detection) equipment necessary. This makes the system more complicated at the first view but has positive aspects on the life cycle cost level in most cases. The longer lifespan, combined with less maintenance, is giving the life cycle cost calculation the right input to end up with lower figures supporting this decision. Middle size turnouts can even be set up in way that the interlocking system sees just one switch machine.

Depending on certain conditions and requirements, different moveable point crossing designs are available.

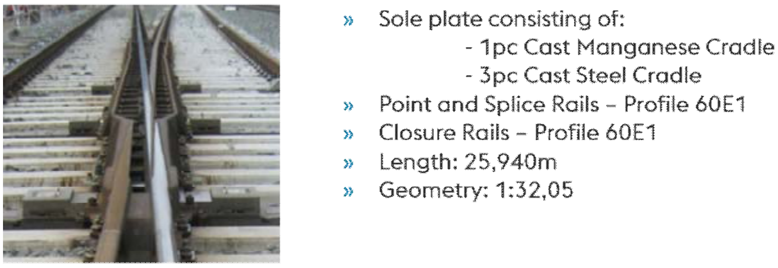


Fig. 11: Moveable Point Crossing with cast a manganese cradle

Fig. 12: Moveable Point Crossing with long wingrails and forged crossing point

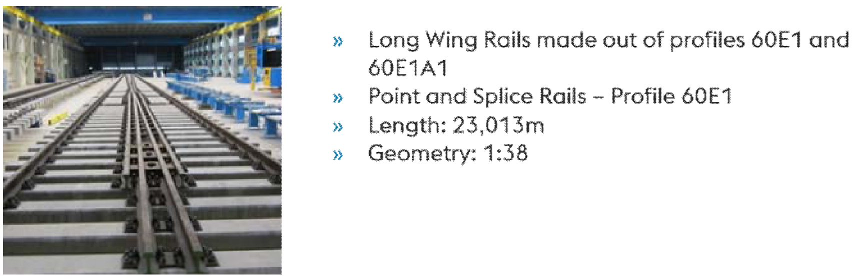


Fig. 13: Moveable Point Crossing with long asymmetric wingrails and main & spliced rail design

Especially for a modern High-Speed system, moveable point crossings guaranteeing safe and efficient operation to the benefit of the end users.

4. BEARING AND ELASTICITY SOLUTIONS FOR HIGH SPEED APPLICATION

The analysis of the damage behaviour of the turnouts in the High-Speed area shows that the ballast is the weakest element in the superstructure. This becomes particularly apparent in the switch and in the crossing of a turnout. Irregularities in stiffness lead to deterioration of the ballast bed, to “hanging sleepers” and finally to strong longitudinal disturbances. A significant increase in the vertical elasticity of the superstructure reduces the static and dynamic loads on the ballast and is thus a possibility on the way to a continuously optimized superstructure (Fig. 14).

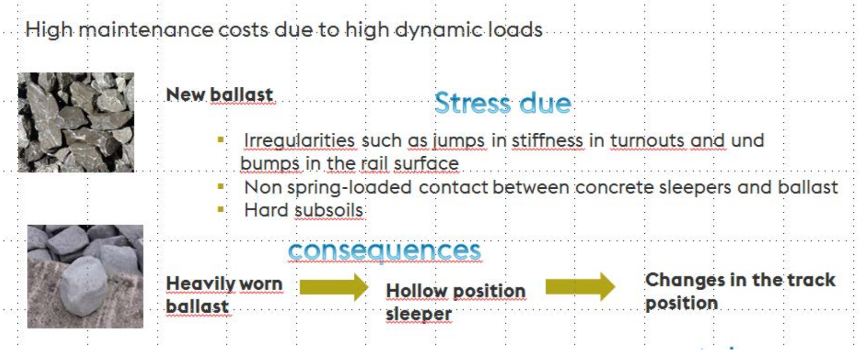


Fig. 14: Ballast as the weakest element in the superstructure

A stiff arrangement of a High-Speed track is not suitable to compensate imperfections of construction and wheels without running into high rates of ballast degradation in combination with high maintenance efforts.

For solving this situation, two strategies can be applied:

- Reduction of the static and dynamic load on the ballast
- Increasing the strength of the ballast

If an elastically mounted rail is overrun by a wheel, the vibration transmission behaviour of the wheel/rail system is determined by the elastic properties of the rails, of the fastening system and the bearers.

The following factors have in particular a major influence on the elastic properties of the system:

- Spring stiffness of the rail bearing arrangement
- Bending stiffness of the rail as a load-distributing beam
- Bearer arrangement and interface to the ballast bed (under sleeper pads)

The mounting of a rail as a continuous beam on many support points leads to a distribution of wheel loads to a certain number of bearers. The extent of the load distribution depends both, on the cross-section of the rail and on the spring stiffness of the support points. If the vertical spring stiffness of the supporting points is reduced, the wheel load is distributed over a larger number of bearers. The bending length of the rail is increased and the loads to the elastic base plates are reduced as well.

The effect of the elasticity of the rail bearing is illustrated schematically in Fig. 15. The diagram shows two linear characteristic curves of different spring stiffness. Furthermore, it is shown how displacement oscillations of wheel/rail with the same amplitudes, mirrored at the characteristic curves, cause different amplitudes of the force oscillations. The aim should therefore be to achieve the softest possible solution (depending on the acceptable rail stresses) in order to minimize the effects of wheel and rail vibrations on the superstructure and substructure (Maurer, Dietze 1996).

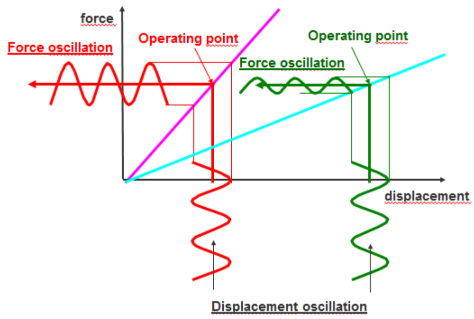


Fig. 15: Influence of spring stiffness to load amplitude (Force oscillation vs displacement oscillation)

Further reduction of the contact stresses to the ballast bed can be achieved by reduced bearer spacings, increased bearer width or/and the use of under sleeper pads. For standard concrete bearers the contact area to the ballast is in the range of 2 to 5% of the total support surface of a concrete bearer. This is resulting in high contact stresses to the ballast causing high deterioration rates finally (Schilder, 2014). To overcome this unfavourable situation, the underside of a concrete bearer is covered with an under sleeper pad. This pad allows the ballast to create a larger and more efficient contact area. In this case the effective contact area to the ballast is in the range of 20 to 35% and reduces the contact stresses to the ballast bed thereof (Fig. 16, 17), (Loy, Augustin, 2015).

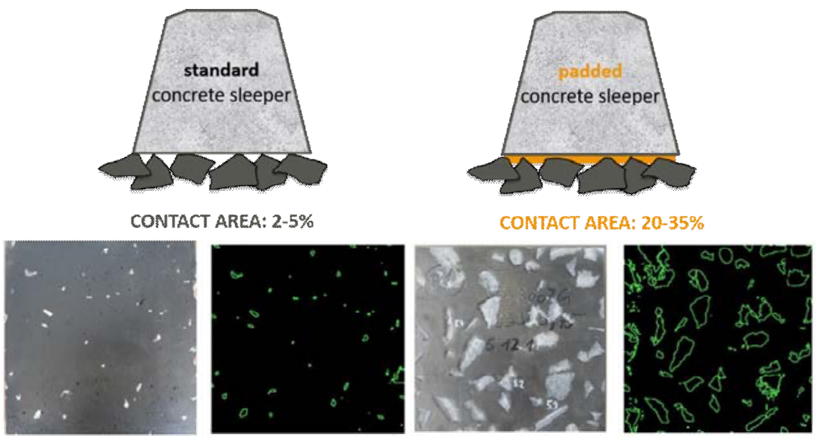


Fig. 16: Under sleeper pads – comparison of effective contact areas

Fig. 17: Under sleeper pads – positive effects, (Schilder, 2014)

In Austria this concept is already used since more than 15 years, also for turnouts, quite successfully. It is a common saying of ÖBB that they cannot afford to have mainline turnouts without under sleeper pads meanwhile. The additional costs are paying back within short due to an increased lifetime of the ballast bed.

As an example of high elastic base plates, the ERL17.5P system will be demonstrated. This system was originally developed by voestalpine BWG for Germany. Meanwhile it is used in High Speed turnouts whole over the world like Germany, Spain, Taiwan, South Korea, China quite successfully.

The rail fastening base plates of the system ERL17.5P are fixed on the concrete sleepers by means of two push-through bolting connections. The direct fixation on slab plates or on concrete bearers supporting is possible too. All ERL base plates are equipped with adjustment inserts in the bolt connections, which enable an easy regulation of track gauge and position errors in the horizontal lateral direction of each ERL plate by +/- 8 mm. An adjustment in vertical direction in the range of -4 mm up to +26 mm is possible. Additionally an elastic pad comprising of sheet steel with vulcanised bearing area (active support springs), cavities and recessed areas (inactive support springs) is placed under a base plate made of steel. The base plates are electrically insulated from the sleepers in vertical direction by means of plastic levelling shims. All base plates of the turnout are configured the same with regard to the fixation on the bearers. They differ in the dimensions and in turn in the adjusted structure of the active support springs of the elastic pad only (Fig. 18), (voestalpine BWG, 2013, 2017).

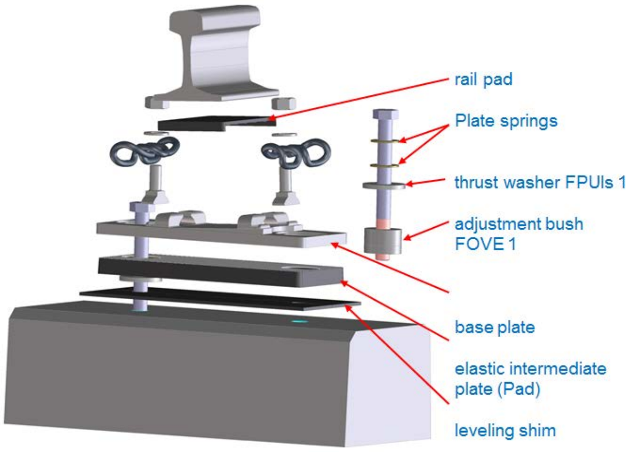


Fig. 18: High elastic fastening system ERL 17,5P

Advantages:

The ideal combination of high-elastic bedding with nearly constant deflection of the turnout's running track on one hand and the horizontal and the vertical adjustability of the ERL on the other hand enables profitable low life-cycle costs for the railway operator in comparison with rigid or less elastic base plate systems.

Highlights of the high-elastic rail fastening base plate system ERL 17.5 P:

- Reduction of forces right at the point of origin
- Easy and fast correction of horizontal and of vertical position errors
- Significant reduction of maintenance costs for the track system
- Applicable with bolted rail fastening systems, e.g. Vossloh Skl12, and with non-bolted rail fastening systems, e.g. Pandrol e-clips or FastClip with and without side post insulators.
- Low maintenance effort

5. RAIL MATERIAL SOLUTIONS TO GUARANTEE HIGHEST SERVICE LIFE IN COMBINATION WITH LOWEST MAINTENANCE EFFORTS

Similar to the efforts to conserve a good initial track quality over a long period of time by keeping the ballast stresses permanently low due to a good distribution of the elasticity in the turnout, this approach can also be used in the area of the wheel rail contact geometry. Preserving good contact geometries can also make a significant contribution to a good long-term behaviour of the track and the switch, and efforts in this regard go far beyond the pure material-related approach to control and reduce damage to the components themselves.

A precondition for good behaviour of rails in the track is a high resistance to deformation of the material in the contact area wheel/rail. A highly deformed microstructure of the material is the starting point for damage of any kind (wear, corrugation and rolling contact fatigue) and at the same time the starting point for plastic flow of material (Joerg et al, 2017). The control of deformation mechanisms is additionally ensuring low wear rates and this is of great importance for ensuring a high profile stability of the rails. In order to combat material deformation, innovative material concepts have been used for some time and have successfully been established in the open track. The newer development is that these steels are now used in the turnout area as well. In particular, the Super Premium rail steel 400 UHC® HSH® from voestalpine (R400HT according to EN13674-1:2017) has proven that its use in the track can realize significant advantages for the operators of railway infrastructure.

The combination of the UHC® steel design (microstructure strengthening) with the HSH® heat treatment technology (microstructure fining) gives this material its special resistance and leads to a considerable improved track performance. The improvement factors achieved by customers in terms of wear, RCF crack development and corrugation are in the order of factor 2 compared to R350HT rails, and the positive effects of maintaining good contact geometries have been documented and confirmed several times (Knoll et al, 2015), (Jussel et al, 2016).

Since fine-pearlitic rail steels have been an integral feature of modern turnouts for decades, efforts were also made to use the highest stage of development of pearlitic rail steels also for turnout construction. Based on many years of positive experience from the open track, basic strength and safety analyses for the production and operation of switches were carried out. For this purpose, numerous mechanical tests and test series for EN/UIC and AREMA application have been carried out in laboratories. These showed that the properties of the R400HT also meet the highest requirements for switches in terms of rail strength, fatigue and toughness. After the completion of the adaptation of manufacturing processes including forging, several switches were produced, which have proven themselves very successfully in the track in Europe and overseas for more than 2 years (Fig. 19 and Fig. 20).

Fig. 19: Heavily trafficked turnout in Austria, Overview

Fig. 20: Heavily trafficked turnout in Austria, Detail

The associated measurement and monitoring programs revealed remarkable performance improvements in terms of damage control and plastic deformation prevention. As an example, the results of measurements (Fig. 21) on a highly stressed curved turnout are given, which were taken after 60 million tons.

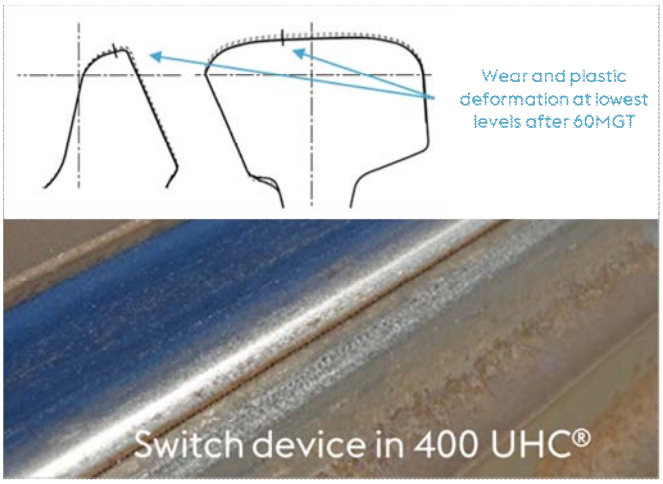


Fig. 21: Optical appearance and measurement results after 60MGT of a R400HT turnout

The low wear rate, the good surface condition and in particular the excellent behaviour with regard to minimal plastic deformation are observed already since the beginning of the observation period. Compared to the steel grade R350HT with approx. 3.5 years service life in the switch blade, a significantly longer service life will be achieved. The positive effects of the 400 UHC® HSH® Super Premium rail steel used for turnout applications thus lead to an optimization of the switches from which innovative Railways in Europe and overseas already benefit.

6. SWITCHING, LOCKING & CONDITION MONITORING

A fast, environmentally friendly and comfortable journey between cities – that is both the wish and need of many people all over the world. Travelling by train is becoming more attractive in relation to air travel, above all on medium-length journeys, thanks to higher speeds and increasing frequencies.

For that reason, especially High-Speed turnouts have to meet the highest RAMS criterions. An unexpected breakdown is causing train delays and many troubles in the system.

In particular switching, locking and detection systems are responsible to direct the train safely into the right direction. However, they have a negative image when it comes down to statistics of unexpected breakdown. In many cases, the statistics show just the last link of the chain highlighting the malfunction. In reality, the situation is more complex. Many imperfections are made in the interface between turnout construction and DLD (Drive, Locking, Detection) system. Both subassemblies must be designed perfectly and must be adjusted properly to each other to work trouble free for a long period.

For an infrastructure company it is the best scenario, when the turnout producer and the signalling producer are working closely together. Therefore, voestalpine VAE has taken this challenge to supply the whole turnout system out of one hand. “One stop shop” means getting all services from a single source.

These guaranties:

- Optimal interface solution between turnout construction and DLD system
- Robust design (turnouts and DLD)
- Highest safety
- Long periods between inspection
- Minimized maintenance requirements
- Longest service life

Resulting in:

- Highest availability
- Lowest breakdown rates
- Lowest LCC (Life Cycle Costs)

In order to fulfil these requirements, voestalpine VAE has developed a comprehensive product portfolio. As an example we will present a High-Speed turnout with single drive, locking and detection points consisting of (Fig. 22):

- SPHEROLOCK® locking points for the switch and moveable point frog
- ECOSTAR switch machines for switch and moveable point frog
- Switch condition monitoring systems for the switch machines
- Endposition detectors for switch and moveable point frogs

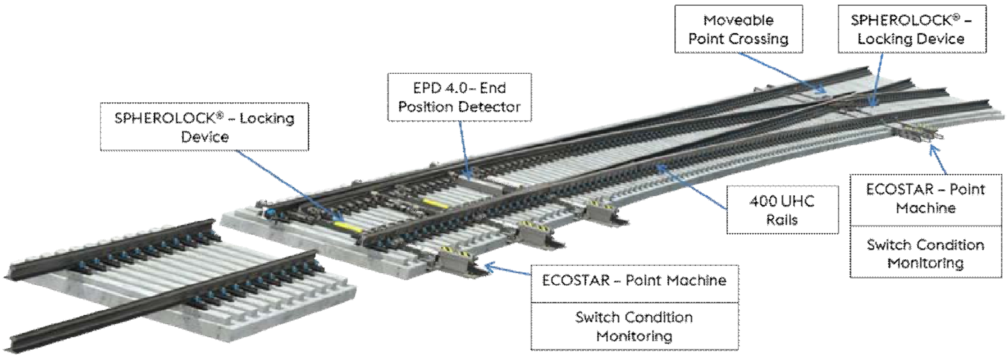


Fig. 22: Integrated Turnout System 60E1-1200-1:18.5 with DLD system

Further improvements on the RAMS and LCC side can be achieved by the use of intelligent monitoring system to provide proper maintenance requirement information in time before repair becomes too costly and my lead to whole component exchange. Furthermore, unexpected breakdowns are avoided, contributing to highest availability.

Fig. 23 highlights the relationship between early problem detection and repair costs. The earlier an arising problem is on the table, the easier and less costly the repair work usually is.

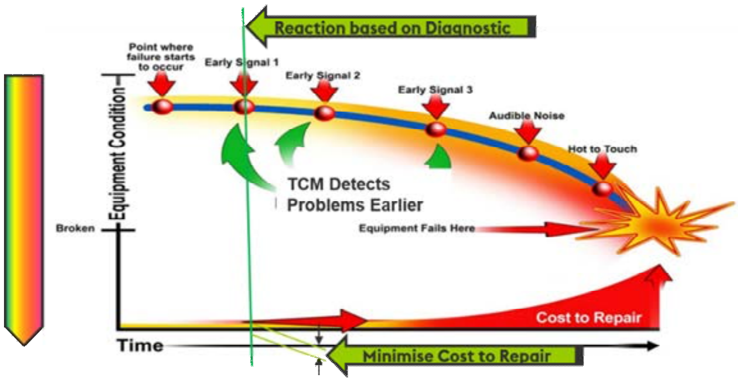


Fig. 23: Relationship between early problem detection and repair costs

Fig. 24 demonstrates the features of an ECOSTAR switch machine with an integrated monitoring and information transmitting system. In that case, the current for switching the turnout, the occurring throw forces or machine oil level etc. is monitored and relevant information is forwarded to a central computer when certain key data are out of a specified range. By doing that, a cost intensive periodic maintenance regime can be substituted by a condition based maintenance regime.

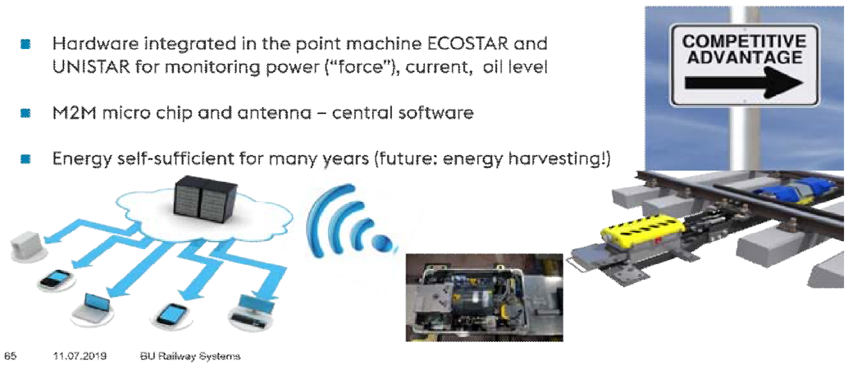


Fig. 24: Intelligent point machine ECOSTAR

Robust turnout designs in combination with intelligent DLD systems, perfectly tuned to each other lead to a very reliable system with reduced life-cycle costs. This is the goal of voestalpine VAE being a design house of completely integrated turnout systems.

7. CONCLUSION

The almost two hundred years of history of railway are characterized by continuous improvements and continuous innovations, which have always been able to keep both vehicles and infrastructure up to an adequate technical and economic level. This characteristic of Railway history ensured and paved the success of the railway to date.

Measures can and must still be taken to reduce the stress on vehicles and the infrastructure. Components, but especially the system, have to be made fit for the future in terms of service life and maintenance intensity, while at the same time being optimised for the present. However, in order to identify and develop measures with the greatest potential for success for the railway, the focus of innovation must be shifted away from the individual components towards the interaction of vehicles with the infrastructure combined with innovative monitoring technologies. This is only possible through good cooperation between industry, science and operators and requires a rethink in the design and design of new products.

Using the example of a vehicle running through a turnout, the potential of this innovative approach can be demonstrated in a very clear way. Changes in the design and construction of the turnout, a kinematically optimized switch blade, or moveable point crossings, lead to considerable improvements here. A well designed elastic bearing of the switch and the use of materials with high damage resistance and profile stability both protect infrastructure and vehicles and permanently from high strains and adjust a desirable condition in the track and the turnout. These measures can make a significant contribution to achieving the objective of overall system optimisation, thereby improving and ensuring the economic efficiency of the infrastructure in the short, medium and long term.

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PERPETUAL PAVEMENT DESIGN, S8 EXPRESSWAY

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SUMMARY

Traditional flexible pavement consists of some (usually three) asphalt layers, where the bottom layer, although loaded by critical tensile stresses is designed and constructed using poor fatigue resistant mix. It is assumed that the stiffer base the lower defection and thus tensile strain at the bottom will lead to longer service life. But the optimum between high stiffness and high fatigue crack resistance is to be analyzed in order to find highest possible durability / pavement life. In this article experiences with design of the first perpetual pavement using anti-fatigue course at the bottom of asphalt layers in Poland are describe. We have to unlearn what we have learned to innovate and to progress.

1. TRADITIONAL ASPHALT PAVEMENT STRUCTURES

Traditional flexible pavement consists usually of three asphalt layers, called from top to bottom: wearing course, binder course and base course. During designing two criterion are taken into consideration: critical strain at the bottom of asphalt layers and critical deformation on subgrade of the pavement. The bottom layer, called also base course, although loaded by critical tensile stresses is traditionally designed and constructed using poor fatigue resistant asphalt mixtures, usually from coarse aggregate and poor binder content. It is assumed that the higher stiffness modulus, the lower defection and thus small tensile strain at the bottom will lead to longer service life. But the optimum between high stiffness and high fatigue crack resistance is to be analyzed in order to find highest possible durability / pavement life.

A National Motorway Construction Program in Poland started at the beginning of this century and the target was to construct missing part of main road network of about 5500 km of motorway (A) and expressway (S) till the end of 2022.

First contracts were constructed at 2001 and typical pavement structure at that time, such as this on Fig. 1, example structure on A4 motorway section was designed in the traditional way. Regularly bellow the asphalt layers the subbase layer consists of so called at time mechanically stabilized (unbound) aggregate sized from 0 to 31.5 mm maximum grain size. Bellow frost protection layer and subgrade. All asphalt layers at that time were constructed using normal bitumen D50, asphalt concrete (AC) and SMA mixes were applied. No modification of binder at that time was taken into account.

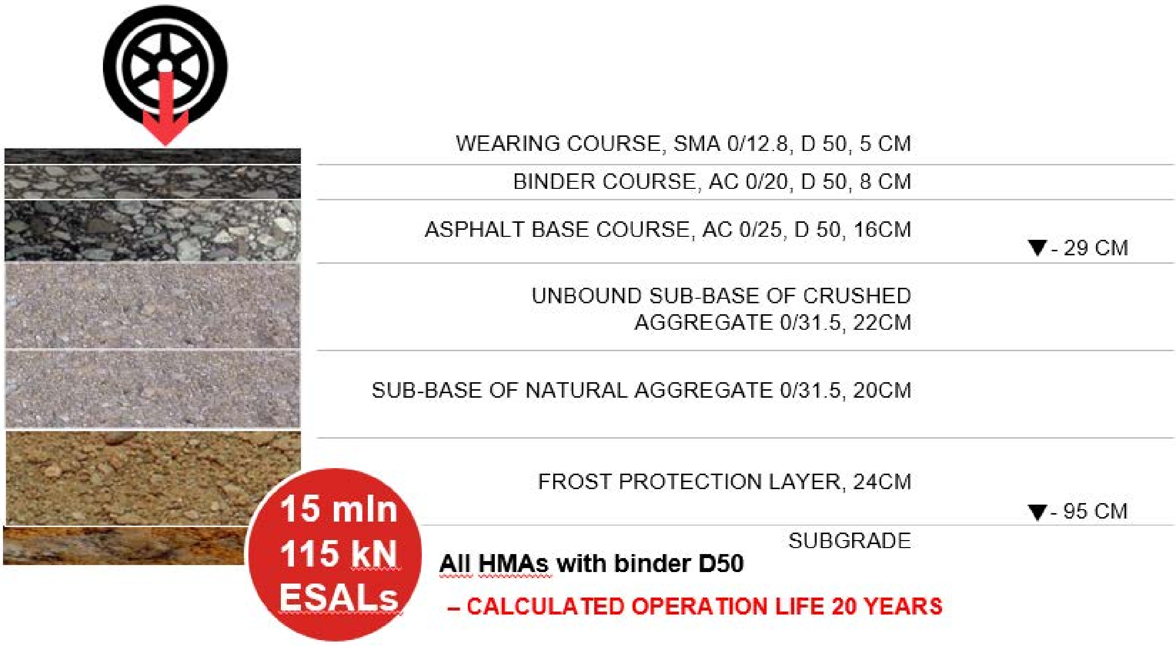

Fig. 1: Typical cross section of traditionally designed asphalt pavement, A4 Motorway, Section Wroclaw - Nogowczyce, constructed at 2001

2. INTRODUCTION OF HIGH STIFFNESS MODULUS ASPHALT MIXES

In 2004 on relatively large scale, but still experimental, section of Motorway A2 Konin - Koło, of the length of about 30 km, was designed and built using for the first time on high traffic volume roads so called asphalt concrete with “high stiffness modulus”, Polish abbreviation ACWMS. Fig. 2 presents a typical cross section of the pavement structure with AC WMS. This kind of asphalt mix was based on French experience with so called high modulus asphalt courses, known in France as EME (Enrobé à Module Elevé), used for base courses. This kind of base course generally differed from the traditional base courses used up to that time in Poland. The binder content was generally higher, air void lower, and the maximum grain size was also lower as traditionally – similar to mixture used for wearing courses. It is assumed that such kind of mixtures, EME type, are generally more durable, resistant to action of water and have better fatigue performance. EME’s mixes in France had maximum grain size usually 14 mm, in Poland it was adopted 16 mm. There was introduced performance requirements in the Contract, such as: minimum stiffness modulus of 14,000 MPa at 10 °C and at 10 Hz and minimal fatigue strain at 10-6 load cycles Ɛ6 of 130 micro-strains, all using 4PB-PR test. These requirements differs from those applied in France where are specified different test conditions and different test method (2PB-TR).

At the beginning there were lot of concerns, regarding relationship between the nominal grain size and course thickness, which was not usual, but first of all due to unusual binder type 20/30, not commonly used in Polish climatic conditions. In France 10/15 is used, but France average yearly temperature is higher than Polish one, so it was decided to use 20/30. Main purpose was to achieve required high stiffness modulus and ensure high resistance to permanent deformation.

Fig. 2: Typical cross section of A2 Konin – Koło motorway, containing high modulus asphalt mix (type EME) in base course, constructed in 2004

A good practical application, compaction ability, smooth and dense surface of base course of high modulus mixes with relatively high binder content and low air void content led finally to fast popularization of AC WMS mixes and its regular application in base as well binder courses on majority of newly constructed motorways and expressways in Poland. Generally two type of binder were applied for these type of mixes: 20/30 and PMB 25/55-60, but with PMB achievement of required stiffness was relatively difficult. Fig. 3 presents standard pavement with prolonged design life contain AC WMS mixes both in base and binder course.

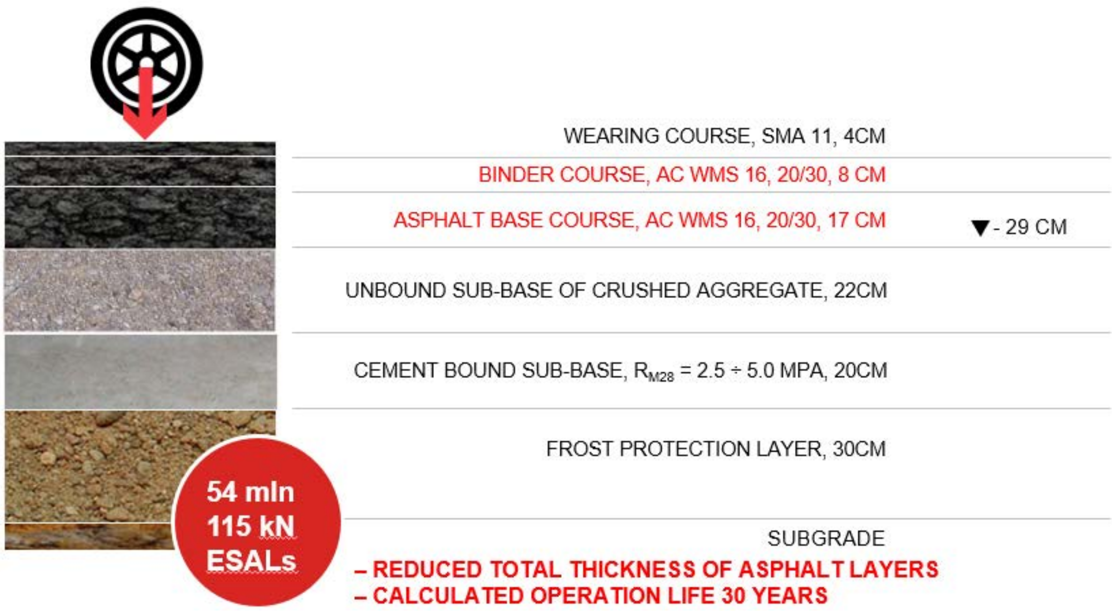

Fig. 3: Typical cross section of A2 motorway, Warszawa – Łódż, with binder and base course with high stiffness modulus mixes (EME type) constructed in 2011

Main purpose of use more stiffer and durable as usual mixes in the pavement structure was to increase the pavement life and reduce the thickness of asphalt layers, however the second target was still not the case, as you see on Fig. 3, where thickness is generally not reduced comparing to standard pavement thicknesses.

3. VERIFICATION OF PAVEMENT STRUCTURES USING HEAVY VEHICLE SIMULATOR

In the meantime in 2008 in the frame of International Research Program called SPENS (Bankowski, 2009) some of the modern pavement structures had to be tested in the different participated ne member European countries. In Poland the partial aim of this project was to verify mainly the efficiency of application of the high stiffness modulus mixes in the pavement structures. A large laboratory performance testing program was carried out and finally field tests using Heavy Vehicle Simulator (HVS) machine from VTI Sweden was executed in Pruszków close to Warsaw. Thus the practical verification had to prove that applying of high stiffness modulus is efficient. The HVS equipment allows to apply the wheel load in accelerated way onto the pavement structure in scale 1:1. So the foreseen lifetime traffic load can be applied to the structure within a few weeks only. Response of the pavement structure can be measured and design assumption verified. HVS machine allows also temperature conditioning of the structure so the accelerated pavement test can be carried out at controlled temperature. During the test in Poland two levels of wheel load were applied, initially standard 57,5 kN and lately increased 80 kN in order to destroy the pavement. In addition to pavement with high stiffness mix comparing to standard, traditional pavement structure TPA proposed to test two additional pavement structures: pavement with semi-rigid composite surface layer of thickness of 5 cm, resistant to deformations, and new pavement type with anti-fatigue course at the bottom of the pavement, using highly modified binder and rich on binder mix. This type of pavement we called mini "perpetual pavement". Mini because whole pavement structure was minimized so that asphalt package had thickness of 12 cm total. Standardized type of unbound aggregate sub-base and cement stabilized sub-grade was applied bellow asphalt layers. Thus four different pavement structures were tested. The Fig. 4 and Fig. 5 present cross sections tested pavement structures and organization of HVS test field in Pruszków.

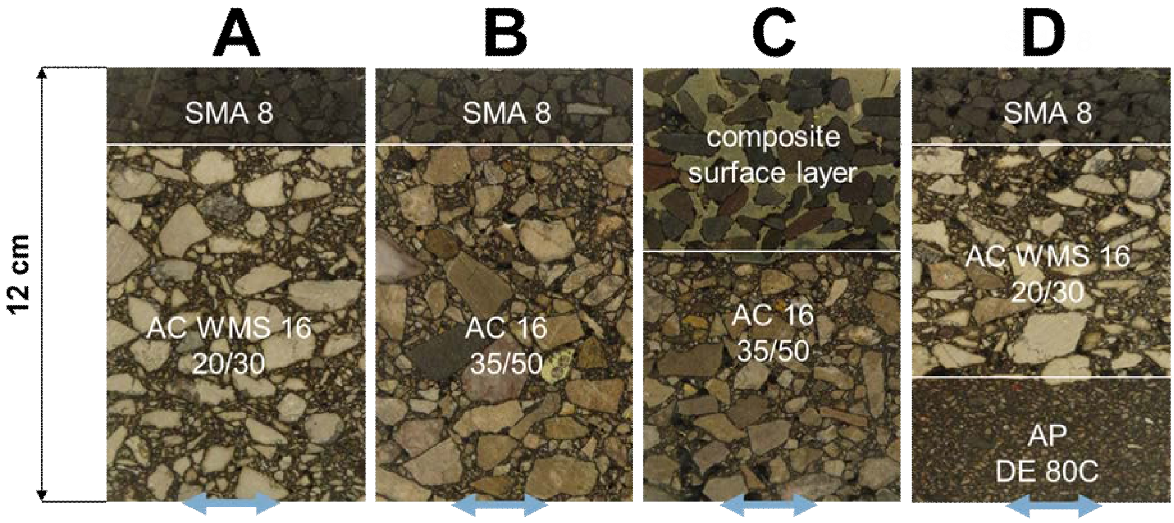

Fig. 4: Four type of the pavements tested in accelerated pavement test in 2008 in Pruszków: high modulus AC WMS base course (A), traditional base course AC (B) – reference structure, composite surface layer (C) and anti-fatigue bottom layer AP – so called mini "perpetual pavement"

Fig. 5: HVS Facility ex VTI Sweden testing among other the first perpetual pavement type structure in Poland in 2008

Thus the first experimental perpetual type of pavement, using anti-fatigue base using PMB instead of traditional AC base course using normal bitumen, was constructed in Poland. It was probably first such type pavement in Europe, as far as we know. Test was organized by Polish Roads and Bridges Research Institute with participation of TPA Poland. Tests field were constructed thanks to STRABAG and Refinery Orlen delivered PMB binder.

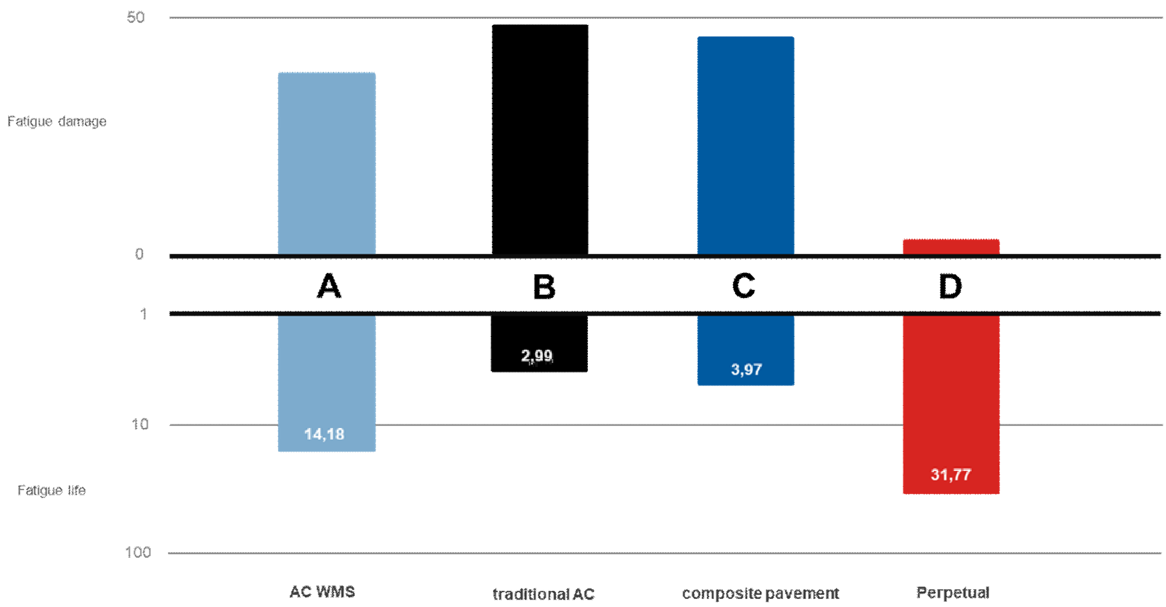

Fig. 6: Fatigue life and fatigue damage of the tested sections (Bankowski,2009)

The main results of the those tests, see Fig. 6, proved that applying the high stiffness mixes into the pavement structure (pavement type A) significantly increase the fatigue pavement life, by minimizing the tensile strain at the bottom of the asphalt pavement. However it was furthermore proved that applying at the bottom of the pavement asphalt layer flexible, not really stiff course instead, with high fatigue resistance (pavement type D) increase the pavement life even more, although the strains at the bottom were higher as in the case of the pavement with stiff base only.

After that test the use of high modulus mixes increased in Poland significantly. On heavy trafficked pavements, especially on motorway and expressway pavements they replaced traditional base and binder courses. Either the penetration 20/30 binder or polymer modified binder PMB 25/55-60 were applied into these type of mixes.

4. PROBLEMS WITH LOW TEMPERATURE CRACKING OF HSM MIXES

During winter in February 2012 air temperatures dropped down to -26 up to -20 °C and kept at this level for about one week long. During a few days of this cold period the low temperature top-down cracks, on full asphalt depth, mainly transversal, occurred on the pavements with high stiffness modulus mixes at almost all regions of Poland. Spacing between those cracks varied from 50 up to some 100 meters. The cracking was mostly observed at the sections where binder 20/30 was used. On the sections where PMB was used the less cracks could be observed and the spacing between cracks was higher. Fig. 7 presents typical transversal low temperature crack on full depth of asphalt layers.

Fig. 7: Low temperature crack on pavements with high stiffness modulus mixes

Some sections were still under construction during this winter and were left during winter uncovered, without wearing coarse. On those sections cracking occurred more intensively, as the surface layer creates additional protection against low temperatures. The application of high stiffness modulus mixes stayed under question mark. Until now they are designed only individually, mainly with PMBs only, as they were excluded from the latest revision of the flexible pavement catalogue. However this was an impulse to the initialization of the next stage of the evolution of asphalt pavement design, finding balance between stiffness and fatigue by reconsidering different binders in different layers.

5. LABORATORY PEFORMANCE TESTING

An extensive laboratory performance testing program on different binders and AC WMS asphalt mixes was carried out at TPA laboratory beginning 2012, to solve the problem of cracking of high stiffness modulus asphalt mixes. As a reference mixtures two commonly applied mixes AC WMS 16 once using 20/30 and once using PMB25/55-60 were chosen. Additionally two special PMB binders with softening point higher than 80 °C: highly modified asphalt binder PMB 45/80-60 (with content of polymers more than 7%) marked in this paper as WM and PMB POLYGUM 45/80-80 binder produced in special controlled process under directed modification marked in this paper as SM. Among other binders normal bitumen penetration 50/70 with addition of special elastomer in order to increase its stiffness in the asphalt mix to the required level was tested too, marker in this paper as DU. All binders were tested using all available test methods in Poland, however the results will not be presented in this article. They can be found in literature reference (Grajewska, 2013). In this article we will focus on selected asphalt mixture performance test results, which are usually of the most importance and direct input data for pavement design, such as stiffness modulus and fatigue resistance. The results of comparative study of performance of AC WMS mixtures type with different binders are presented in Fig. 8. Tests were carried out in 4PB-PR method, which is standardized in Poland. Stiffness is measured at 10 °C and at frequency 10 Hz. Fatigue parameter Ɛ6 is represents a strain at million cycle derived from fatigue curve obtained at different strain levels, at 10 °C during 4PB-PR controlled strain fatigue test.

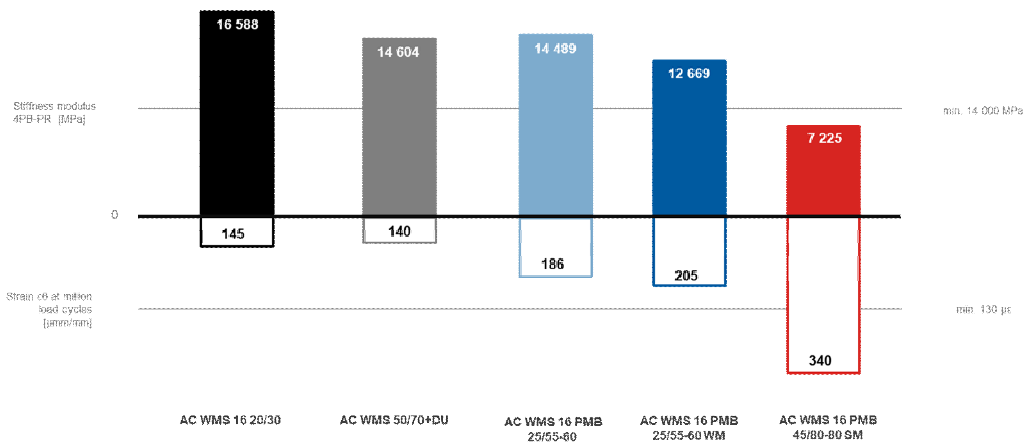

Fig. 8: Stiffness modulus vs. fatigue resistance of AC mixes with different binders obtained at 4PB-PR tests, on the right side the requirement put on high modulus stiffness mixes in Poland

A special attention needs a fact that specially modified binder PMB POLYGUM 45/80-80 showed most superior properties concerning fatigue behaviour, as well as high resistant to permanent deformation and highest low temperature cracking resistance. What was but most surprising that practically this type of binder showed no aging comparing to other binders and binder systems. Even recently highly modified binder which is increasing on popularity of use had no such excellent properties as this kind of binder.

When we look closer to decrease of stiffness modulus during fatigue test in Fig. 9 comparing three different binders, we see that mixes with binders with relatively high stiffness modulus losing the stiffness much faster than binder which have initially much lower stiffness. Based on that observation as well, we could assume that high stiffness is not always of benefits concerning durability of the asphalt mix and thus pavements. So the not only modulus but the tempo of it decreasing under repeated load should be of importance considering different mixtures containing different binders. Figure presents results for AC WMS mix with three different binders. Arrows show load cycle where stiffness dropped to 50 % of its initial value, considering as the end fatigue life.

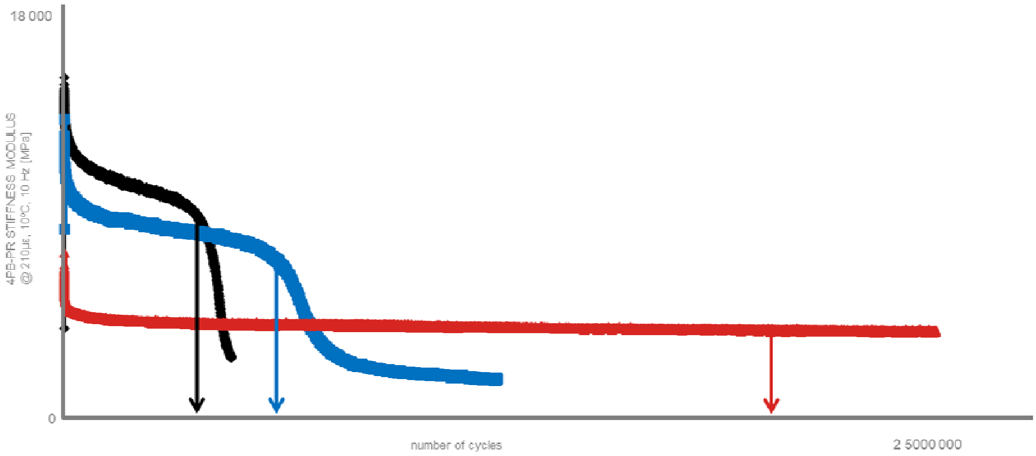

Fig. 9: Decrease of AC mix stiffness modulus during fatigue test for three different binder types: 20/30 (black), PMB 25/55-80 (blue) and special one PMB POLYGUM 45/80-70 (red)

Based on these findings a simulation of pavement design using laboratory obtained performance parameters of two mixes (asphalt concrete with high stiffness modulus AC WMS), differing by kind of applied binder only, was carried out. One mix “stiff” with “worse” fatigue resistance contained binder 20/30 and the other “flexible” with “better” fatigue resistance contained PMB 25/55-60. Typical pavement structure for traffic load class 5 was taken to simulations. Total thickness of asphalt package was 24 c. Wearing coarse remained in all cases the same. Variable was placement and thickness of those two mixes with two different binders which are normally according to catalogue equivalent. For the combinations presented at the Fig. 10 pavement life was calculated based on laboratory parameters of those two kind mixes, using French design method, utilizing results of 2PB-TR tests, for which the design method was calibrated. Two criterion were taken into consideration when calculating pavement life, once critical tensile strain at the bottom of asphalt mixes (red columns on the figure) and once critical permanent deformation on the subgrade (blue column).

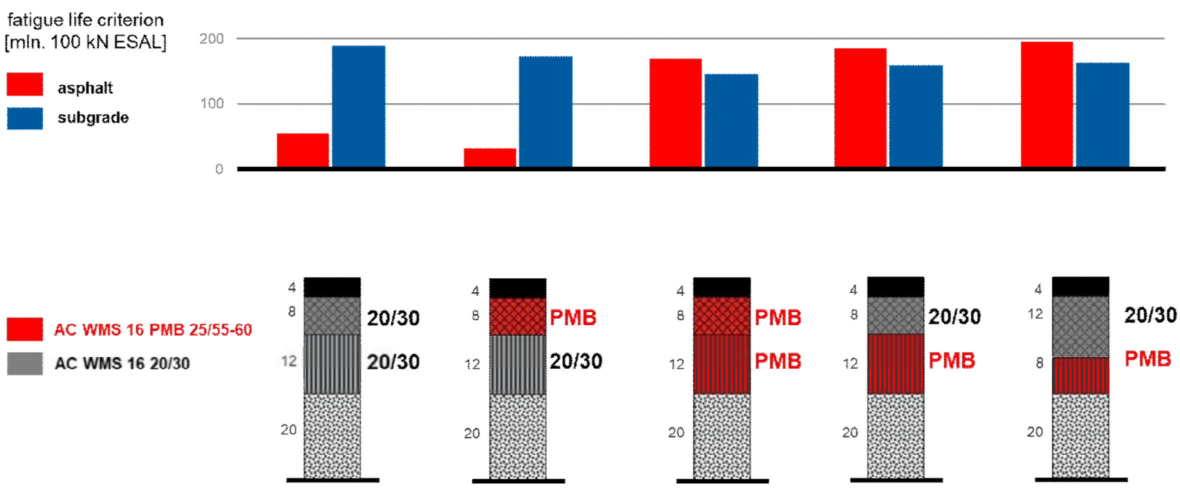

Fig. 10: Comparison of fatigue lifetime calculated based on laboratory performance data of two mixes with different stiffness and fatigue resistance, varying their location (between binder and base course) and thickness in the pavement structure

Analyzing the results presented on Fig. 10 it was proven that principally the highest possible pavement life can be achieved when: the base (bottom) course is made of "flexible" asphalt mix with improved fatigue resistance, containing PMB binder, which usually guarantees such properties, and the binder (middle) course is made of "stiff" mix. However the thickness of base course shall be optimized, reduced to necessary minimum in order not to "weaken" the whole structure, because when we would use regular thicknesses of stiff base courses this would not give us the possible fatigue life and would be additionally economically ineffective.

These analysis carried out in order to optimize the pavement design with asphalt mixes with high stiffness modulus led finally to the conclusion that best possible fatigue life can be obtained applying the highly resistant asphalt mixture, relatively rich at the binder content of high quality asphalt binder, such as e.g. PMB, at the bottom of the pavement structure - in the place where tensile stresses can occur under traffic load. Conceptions of such pavements are also know from literature (e.g. Newcomb, 2010).

6. PERPETUAL PAVEMENTS WITH ANTI-FATIGUE BASE COURSE

Perpetual pavement is according to definition of APA (Asphalt Pavement Alliance) „An asphalt pavement, designed and built to last longer than 50 years without requiring major structural rehabilitation or reconstruction, and needing only periodic surface renewal …".

One of the ways how to design such pavement is to apply so called "anti-fatigue" base course at the bottom of the asphalt layers and high stiffness modulus mix at the middle part of the asphalt package. As demonstrated on Fig. 10, the thickness of "elastic", e.g. not too "stiff", anti-fatigue course has to be designed so that is covers all possible tensile stresses occurring under load at the bottom of the pavement, remembering that subgrade has to be improved as applying less stiff material at the bottom we increase pressure on the subgrade.

Fig. 11 explains a fundamental difference between traditional and perpetual pavement structures. At the area of highest compression and shear stress, generally from 10 to 16 cm bellow the surface, a "stiff", resistant to permanent deformation mixes, such as EME type mixes, ensuring sufficient low temperature resistance (depending on climatic conditions) has to be designed. The tensile stress area at the bottom which is generally from 7 to 10 cm, depending on design traffic load, has to be constructed using material with high fatigue resistance, not necessarily stiff. General idea of perpetual pavements is by the way reduce the critical strain at the bottom of the asphalt layers to values bellow 70 micro-strains, so called endurance limit (Newcomb, 2001), what should ensure that pavement is working at such small strains that it will theoretically never fatigue damaged. However as shown earlier in this paper and in author's opinion initial high stiffness modulus does not guarantee long lifetime as during repeated load can decrease very significantly. This depends however strongly on asphalt binder quality. If we have very high quality binder we can allow even higher stresses at the bottom of the pavement.

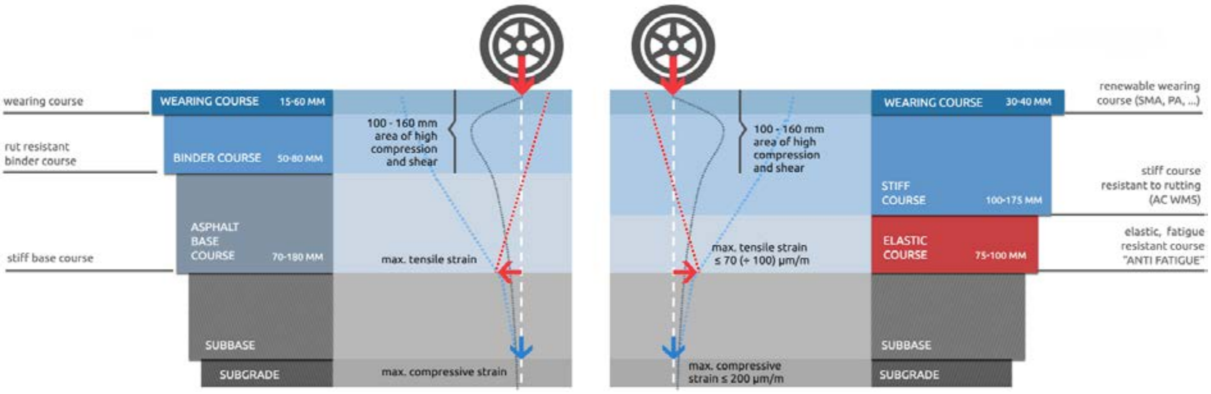

Fig. 11: Comparison of traditional pavement structure and perpetual pavement structure with anti-fatigue bottom asphalt course

7 PERPETUAL PAVEMENT DESIGN ON EXPRESSWAY S8 OPACZ – JANKI

One of the first Design & Build projects in Poland was S8 Opacz – Paszków, close to Warsaw, tendered by public investor GDDKiA in 2013. This was the chance for contractor (STRABAG) to propose at the design stage an alternative proposal to the traditional pavement structure design, which was assumed to be designed in the conditions of the contract, where one condition was to design total thickness of the pavement of 31 cm.

Fig. 12 shows the pavement structure which could be designed and approved when traditional approach would be chosen, with total asphalt thickness of 31 cm, comparing to finally approved proposal of the perpetual asphalt pavement, with total asphalt thickness of 26 cm with simultaneously improved subgrade in order to avoid early subgrade damage.

Reducing the thickness of the asphalt package by 5 cm and improving the subgrade pavement design lifetime was increased from 32 million of equivalent standard axle loads (ESAL) of 100 kN up to 142 million. For the middle part of the asphalt layers a high stiffness modulus asphalt concrete (more than 14000 MPa at 10 °C and at 10Hz) mix AC WMS 16 using PMB 25/55-60 was designed, with additional requirement on low temperature cracking resistance by TSRST method of lower than -22 °C. There were many different approaches for so called anti-fatigue layer, but finally an AC 16 mix using special binder PMB POLYGUM 45/80-80 was found to be the ideal solution for anti-fatigue course, as due to its excellent fatigue resistance, at 4PB-PR fatigue test results showed ε_6 strain at level of 400 micro-strain. More details about design assumptions and test results can be found in (Ruttmar, 2016).

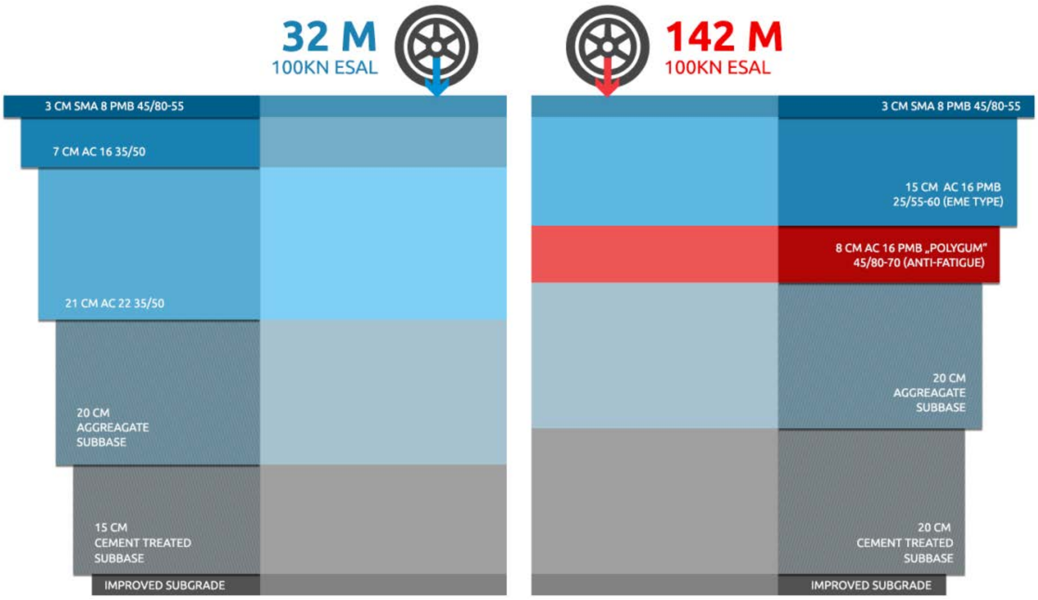

Fig. 12: Traditional pavements structure, tendered on S8 expressway Opacz – Janki (left) and proposed alternative perpetual asphalt pavement (right), constructed in 2015

First trail section of anti-fatigue base course in the thickness of 8 cm, of the length of 150 m was constructed in September 2014 using regular paving equipment. Compaction of the layer is presented on Fig. 13 Special binder was imported to the mixing plant from abroad as at that time no such binder on Polish market was available and/or sufficiently tested.

After quite long process, where among others contractor was obliged to accept extended guarantee period, submit to Employer complete maintenance program and life cycle cost analysis and two independent experts opinions, the first (according to author's knowledge first in Europe) perpetual pavement with anti-fatigue base course was approved an construction completed in 2015.

Fig. 13: Construction of trial section of "anti-fatigue" bottom asphalt layer in the thickness of 8 cm, S8 expressway Opacz – Janki, 2014

8. CONCLUSIONS

Traditionally the flexible asphalt pavement structure consists of more, usually three layers, where the bottom layer, also called base course is usually designed as the thickest and stiffest one, containing coarse asphalt mixes with relatively low content of normal bitumen. Such mixes, although relatively stiff have usually poor resistance to fatigue. But when we design at the bottom of the asphalt layers more elastic and fatigue resistant course, so called ant-fatigue course, we can prolong the pavement significantly, e.g. by factor 2 or more. So finally we can design the pavement which are fully sustainable, economically and technically much more efficient and beneficial for the society.

Sometimes we have to just unlearn what we have learned (traditional pavement structure design) in order to innovate and to evolve to the next stage of asphalt pavement designing. We have to put right material into right location in the pavement.

9. ACKNOWLEDGEMENTS

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A special thanks to GDDKiA - General Directorate of the National Roads and Motorways, Department of Technology and Headquarter office in Warsaw for approval of the proposal, for support to make it formally happened and finally for the courage to change the traditional way of designing in order to develop asphalt pavement design to the next stage.

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RAIL DIAGNOSTIC DEVELOPMENTS

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SUMMARY

By the increase of the requirements against railway infrastructure and by the increase of the loading of the track railway experts recognised that the state of the railway track and its changing must be regularly examined by up-to-date diagnostic devices.

One of the decisive elements of the railway track structure is the rail. The wheel-rail contact basically influences the quality of the rolling which has an effect on the safety of the traffic, deterioration of the structures, their lifetime and maintenance.

In the past period the increase of the axle-load and the development of the vehicles brought the formation of Rolling Contact Fatigue „RCF" rail defects appearing at rail-wheel connection and originating from rolling contact.

1. WHAT KIND OF DEFECTS SHOULD BE REVEALED IN THE COURSE OF DIAGNOSTIC EXAMINATIONS?

Railways generally started to pay special attention to the examination of rail defects on the effect of some great accidents occurred.

In the beginning these examinations were made by manual devices. The first rail diagnostic trains appeared in the 1950s.

For diagnostic experts diagnosis of rail defects means a great challenge.

In the frame of the presentation the results of the newest developments are presented.

The railway safety and the economical operation demand the continuous development of rail diagnostics.

2. RAIL DIAGNOSTIC EXAMINATIONS

The different rail test procedures developed parallel with the development of railways. In the beginning there were only some visual and simple tests. Nowadays the state of the rails can be determined by very modern and difficult test procedures.

Inspection methods presently applied can be divided into two substantial groups:

- Examination of the inner characteristics
- Examination of geometrical and surface characteristics of the rail

Examination of inner characteristics of the rail can be further divided into the following tests:

- *Ultrasonic rail inspection.* This test method has already been applied by the railways since 1950s. It's true that at that time a very primitive test procedure was applied. Nowadays these tests are made by very difficult inspection trains.
- *Rail stress measurement.* The exact knowledge of the thermal forces arose in the rail is essentially important for the operators.

Also at the examination of the *geometrical and surface characteristics of rail* several test procedures can be listed:

- *Rail profile measurement.* It means the measurement of the cross-section parameters of the rail, relatively simple procedure and it can be made with great speed.
- *Rail corrugation measurement.* With the increase of the speed the measurement of this parameter received a greater and greater role.
- *Eddy Current examination.* Originating from the rail and wheel connection rail head cracks (head checks) appeared in the last decade.
- *Examination of the surface faults of the rail.*

From the above listing it can be seen very well that very serious diagnostic tests are available for the determination of the state of the rail.

Lest we should forget that the knowledge of the state of the rail is an important safety question. Furthermore at the designing of the maintenance works it is the primary aspect that we should have appropriate data about the deterioration processes.

3. DEVICES AND EQUIPMENT OF RAIL DIAGNOSTIC MEASUREMENTS AND EXAMINATIONS

3.1. Ultrasonic inspections

Ultrasonic (US) rail inspection is an important element of the track diagnostic system whose aim is to find such defects in rails and rail welds, not visible by eye, which can cause later the brake of the rail.

For the sake of the operational safety of the track these examinations should be made by regular intervals (in case of need immediately) in order that we could pay attention to the continuously changing defects, and we could replace the faulty rails and rail welds in time.

Manual ultrasonic examination devices

Hungarian State Railways has been dealing with the ultrasonic examination of rails since 1959. In the past period from the beginning we developed manual ultrasonic testing trolleys, by which we primarily do continuous rail tests.

Fig. 1: Manual ultrasonic examination devices

Testing devices have 3-6 channels, by radiation of 0 degree (perpendicular), of 70 degrees and of 45 degrees. By this arrangement the mostly loaded part of the rail cross-section is detectable efficiently.

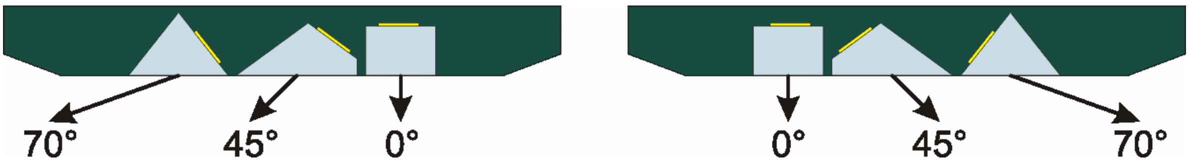

Fig. 2: Location of examination probes

Mechanical ultrasonic inspection

The ultrasonic rail testing systems are located on MÁV CRTI Ltd's rail diagnostic train-set (SDS) and on the newly developed motor train (FMK-008). The measuring trains can travel in both directions.

Mechanical ultrasonic inspection consists of two main phases, from the mechanical examination itself, and from the field manual post-examination of the defects which need provisions. The defects requiring provisions must be made more accurate on the spot by manual post-examination (chainage /sections/, defect qualification, signing).

Fig. 3: SDS rail diagnostic measuring train (1997)

Generally several measuring systems complementing each-other are installed on measuring trains. The following measuring systems are placed on the measuring trains of MÁV Central Rail and Track Inspection Ltd:

- Ultrasonic inspection system
- Eddy-current measuring system
- Rail corrugation measuring system
- Rail profile measuring system
- Video system

Fig. 4: FMK-008 rail diagnostic measuring train (2016)

Fig. 5: Special measuring bogie (for ultrasonic, eddy-current and rail corrugation measurement)

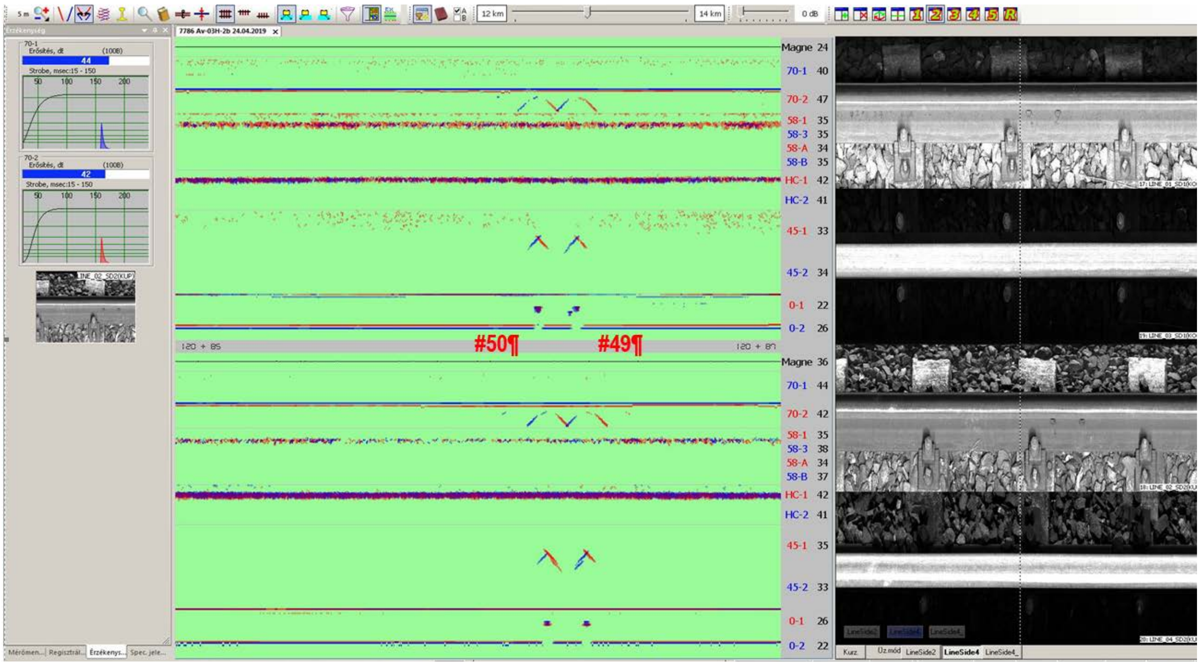

Fig. 6: US evaluating image

The designable maximum testing speed of the examination on a track of good quality is 70 km/h, on a track of bad quality and on traditional jointed track is 30 km/h.

3.2. Rail stress measurement

Determination of the thermal originating stress and the neutral (stress-free) temperature emerging in rails is basically important task.

Determination of neutral temperature can be listed into two main groups:

- *Destructive method.* By cutting the rails the neutral temperature of the track can be determined very easily. Generally the neutral temperature of the track is calculated from the displacement value of the rail.
- *Non-destructive methods.*

The neutral temperature measuring system (Rail Stress Diagnostic = RSD) came into existence from the collective application of two inspection methods. The simplification of the measuring process was successful by the application of a mechanical (SidePull) and a magnetic (RailScan) methods. Of course both methods can be used respectively as well.

Fig. 7: RAILSCAN device   Elements of SidePull measuring system

3.3. Rail profile measurement

Transversal profile is measured by contact-free, laser technology even at a speed of 80 km/h.

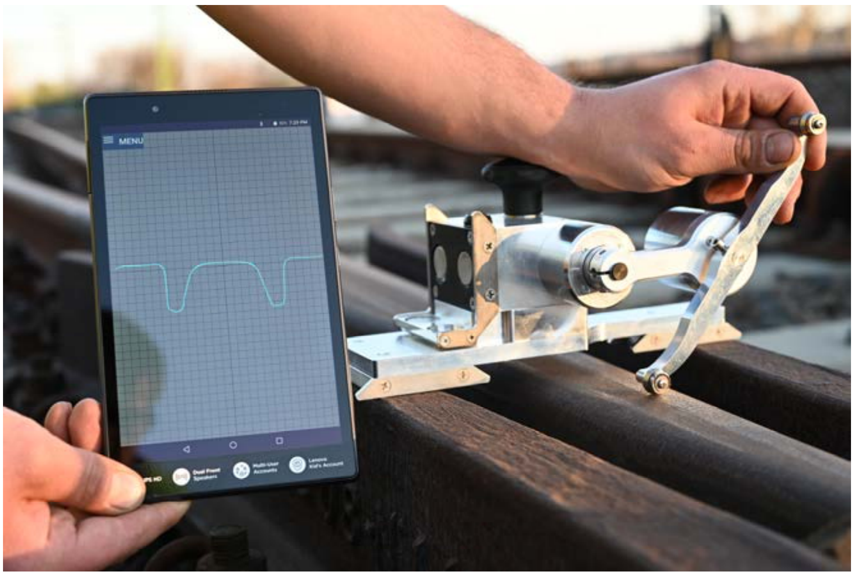

Fig. 8: Handheld rail profile meter

The mechanical rail-profile measuring system possesses with an automatic rail recogniser, on the base of which it supplies the lateral and vertical wear data.

For the measurement of the rail profile a lot of manual and mechanical measuring systems can be applied.

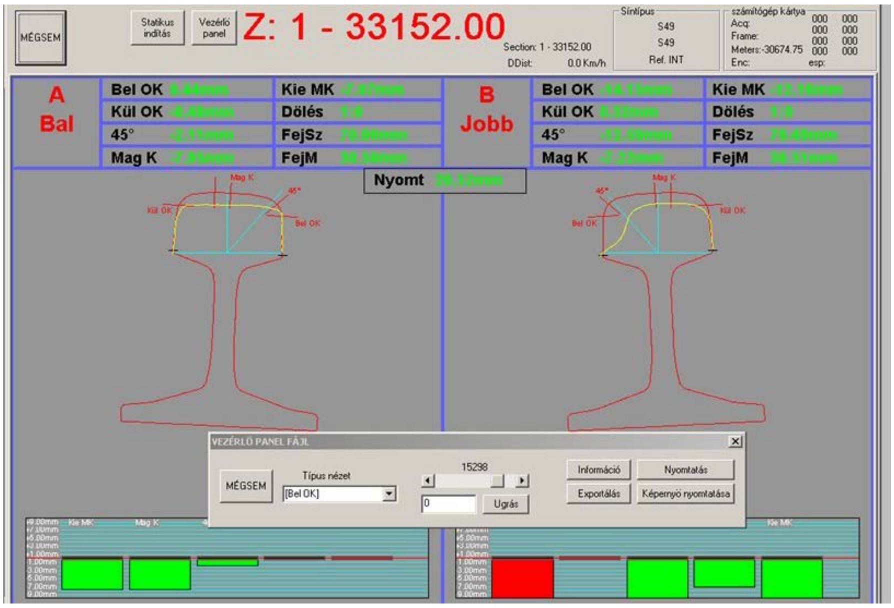

Fig. 9: Measurement of the rail cross-section

3.4. Rail corrugation measurement

Corrugation of the rail surface is measured by induction principle, then the wave length and amplitude are determined by calculation, on the base of which rail grinding is designable.

Fig. 10: Corrugation measurement graph

3.5. Eddy-current inspection

Fatigue damages, rail defects caused by Rolling Contact Fatigue (RCF) appeared in a great extent on the rails lately.

Fatigue damages caused by the rolling contact, i.e. the checks arising in the rail head (Head Checking) mean those check formation which originate on the effect of great sliding forces (especially the great longitudinal sliding forces) and by the material displacements close to the rail surface.

The main factors in head checking damage seams to be the high contact stresses between wheel-rail and a small sliding between the wheel surface and the rail.

3.5.1. Appearance forms of Rolling Contact Fatigue (RCF) Head-checking

Types of defects arising from rolling contact

Rail head damages arising during track operation. By the appearance of modern and heavy-duty hauling vehicles the (Rolling Contact Fatigue = RCF) rail head damages arising from rolling contact increased like jumping. The challenge of these days that we could determine these defects with appropriate safety and punctuality.

Head checking: Rolling fatigue crack appears mainly on the guiding surface of outer (higher) rail of the curve which is caused by the material fatigue due to the high dynamic loads of wheel-rail connection.

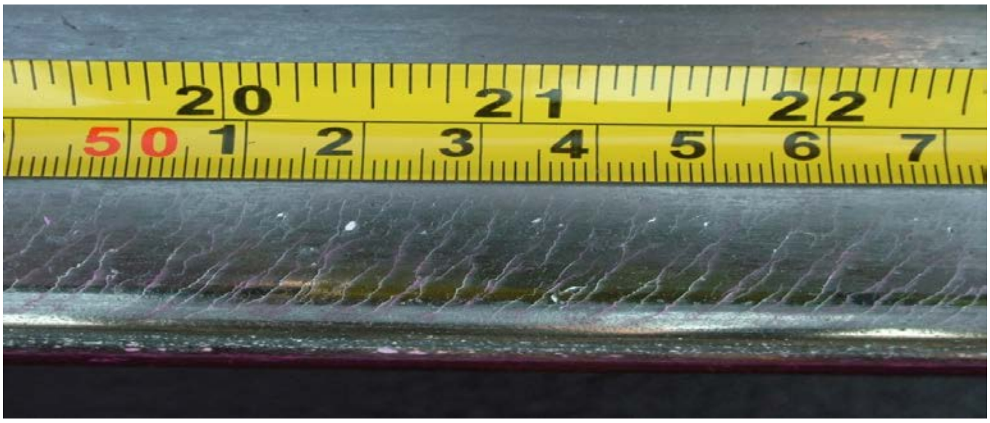

Fig. 11: Head checking

Rail head compression (Squat): Individual cracks of flat angle under the running surface in V-shape (legs of the V open towards the guiding surface), which cause brownish colour dishes as time goes on.


Fig. 12: Rail head compression (Squat)

Tongue lipping. Tongue lipping is generally frequent in curves where surface cracks emerge on the contact point of wheel flange then tongues are formed starting from the cracks due to the effect of wheel flange.

Fig. 13: Tongue lipping

Crack nest (Belgrospi). Crack nest always arises in the valleys of vibrational corrugation due to material fatigue, and it is inseparable from the phenomenon of corrugation.

Fig. 14: Crack nest (Belgrospi)

Imprint. Rail defect arising by sticking an individual exterior object (e.g. piece of stone) between the wheel and rail.

Fig. 15: Imprint

Vibrational corrugation. Corrugation arising in straight tracks and in both rails of curves of high radius which is caused by material changing due to the vibrations of wheel–rail connection.

Fig. 16: Vibrational corrugation

Places of wheel sliding. Material hardening, material flow, indent, etc. caused by spinning-off, blocking arising on acceleration and braking sections.

Fig. 16: Places of wheel sliding

3.5.2. Eddy-current measuring systems and devices

Lately the manual inspecting instruments were developed according to the demands. Basically it is determined by the fact, that how many probes the instrument has. There are instruments of one-, two, four and eight probes.

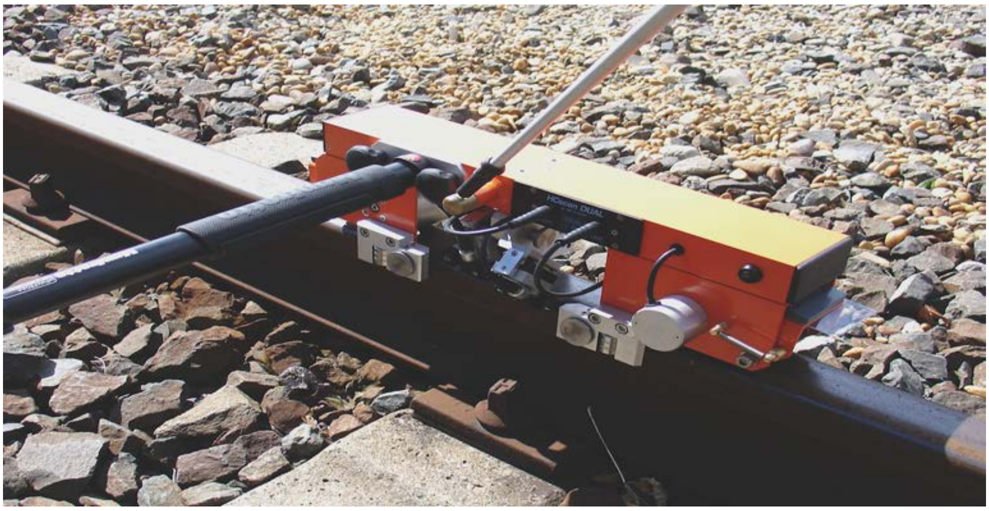

Fig. 17: HCscan-dual

Fig. 18: GF08 Rolling Contact Fatigue Crack Measuring Device

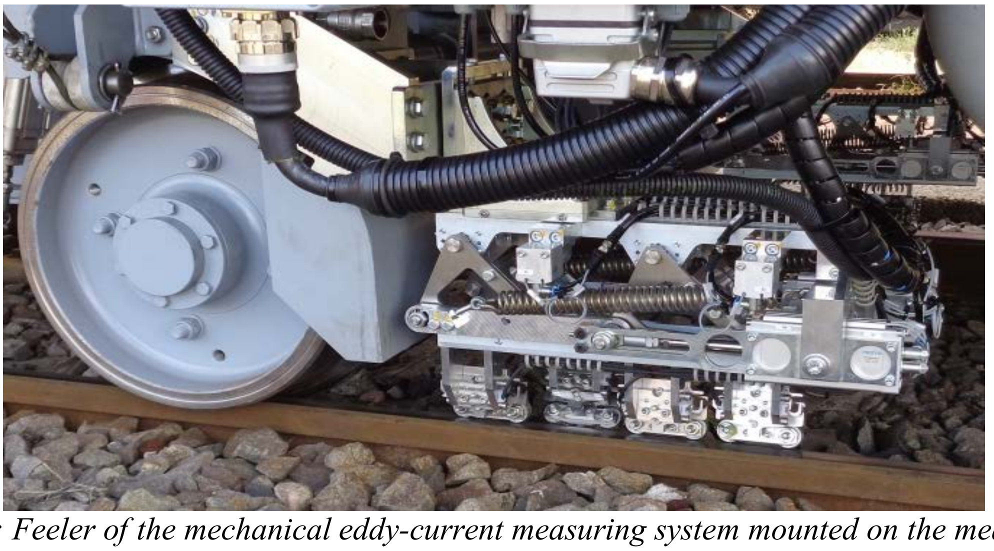

Fig. 19: Feeler of the mechanical eddy-current measuring system mounted on the measuring bogie

Fig. 20: Result of the eddy-current test

3.6. Examination of the rail surface faults

The rapid technical development in the imaging enabled that the surface defects of the rail can be recorded by high-resolution video cameras. After the evaluation of images we receive the defects of the rail surface. By this system only the fully-developed HC defects can be detected. HC defects in the initial phase cannot be discovered by this system.

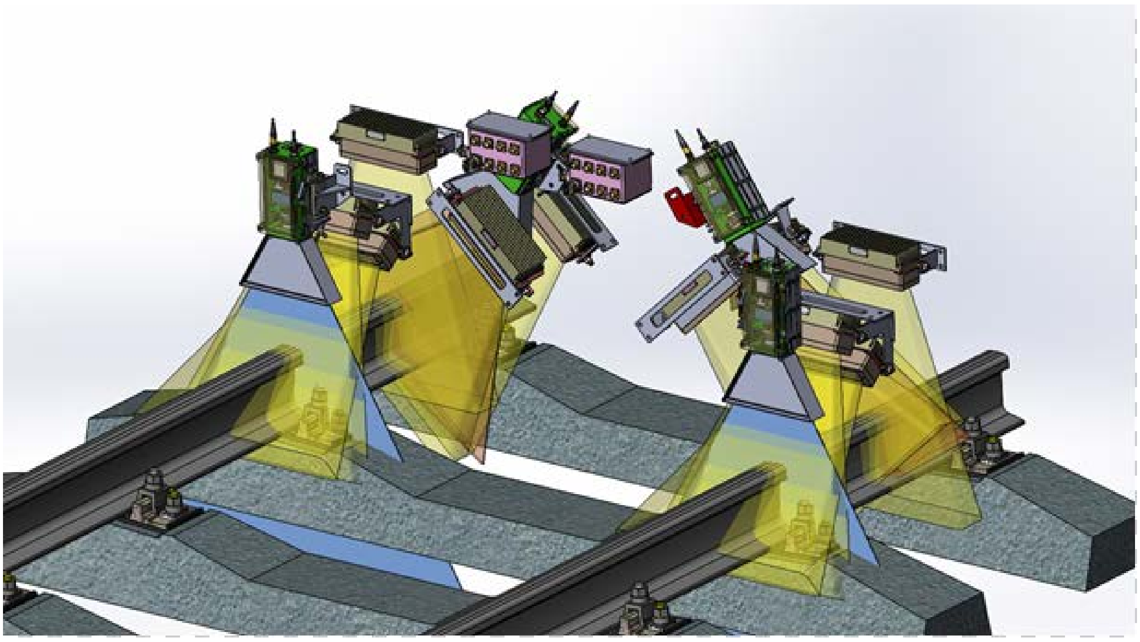
Fig. 21: Position of the linear video cameras

Fig. 22: Video system on MÁV CRTI Ltd's rail diagnostic measuring train

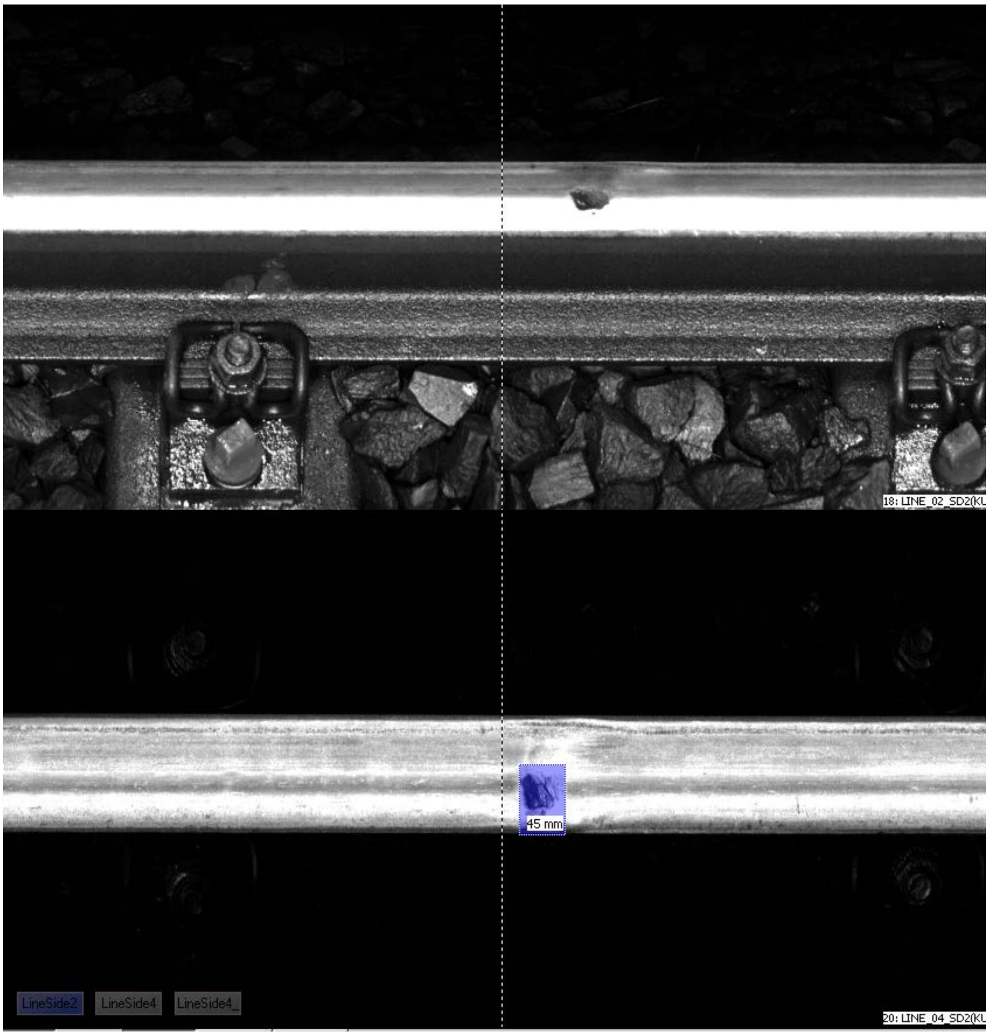

Fig. 23: Video image of a surface damage fault

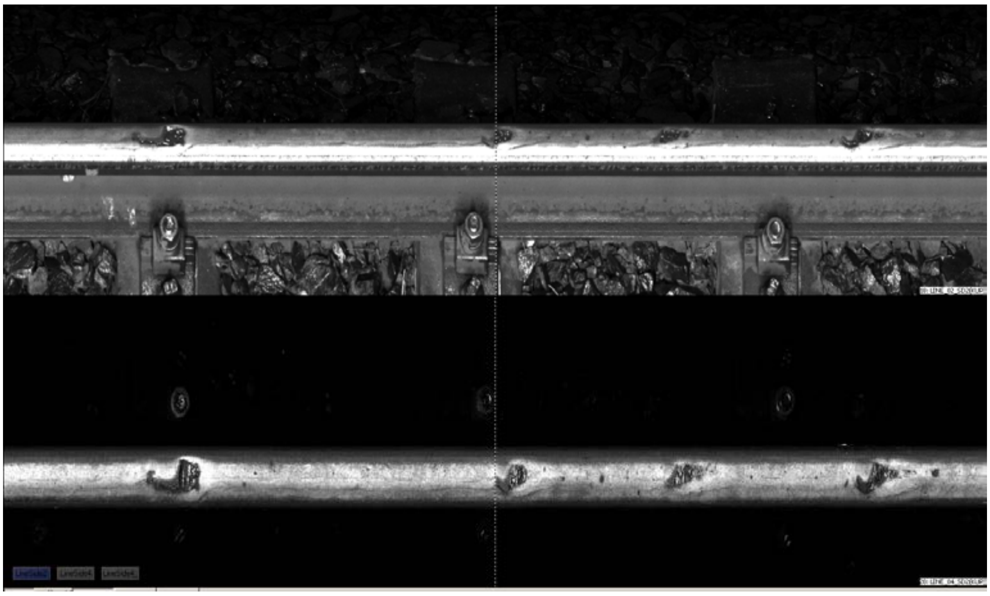

Fig. 24: Rail surface damages

4. SUMMARY

The result of the development of rail diagnostic systems is that we can determine the state of the rails more and more accurately, by this the safety of railway transport increases and the changing in the state of rails can be continuously traced. Furthermore the knowledge of the state gives help for the accurate designing of rail maintenance and for the execution of necessary works in time.

The railway safety and economical operation demands the continuous development of rail diagnostics.

ANALYSIS OF CRUSHED BALLAST PARTICLES UNDER LABORATORY MEASUREMENT CONDITIONS

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SUMMARY

This paper contains some conclusions from the author's research that the authors experienced during their individual fatigue laboratory tests.

The most railway lines in the world have so called traditional superstructure (ballasted tracks). In the past few years there were a lot of railway rehabilitation projects in Hungary, as well as abroad. The authors think that it is important to learn about the process of ballast degradation, because ballast material is the largest weight in the track.

The authors' research's main goal is to be able to simulate the stress-strain effect of ballast particles in real and objective way in laboratory conditions as well as in discrete element modelling.

The authors have developed two types of individual laboratory tests.

As the second laboratory test, the use of the available laboratory test tools has recently started as a new direction for the research topic. The authors worked out two types of test methods: one of that has wide range of literature, but for the other one there is no relevant source, yet.

Hopefully, adequate new results can be achieved with developing the new methods in the research topic.

1. INTRODUCTION

An article was published (Fischer, 2015) in 2015 with results of an R&D on individual breakage test method in laboratory related to railway ballast. Since that several other publications were published in this topic (Fischer, 2017; Fischer, Németh, Harrach, Juhász, 2018; Fischer, Németh, 2018a; Fischer, Németh, 2018b; Juhász, Fischer, 2018; Juhász, Fischer, 2019a; Juhász, Fischer, 2019b; Juhász, Fischer, 2019c; Juhász, Fischer, 2019d). The authors in this paper would like to present the research topic with its laboratory measurements.

The rock physical suitability of railway ballast materials is determined by laboratory tests in the EU, formulated in the same product standard.

There are two types of standardized tests in the aspect of rock physic characteristics of railway ballast:

- Micro-Deval abrasion test according to MSZ EN 1097-1:2012 (MSZ EN 1097-1, 2012);
- Los Angeles abrasion test in accordance with MSZ EN 1097-2:2010 (MSZ EN 1097-2, 2010).

These are determined in the MSZ EN 13450:2003 (MSZ EN 13450, 2003) product standard.

These two test types are absolutely suitable for satisfy defining the abrasion characteristics of a given aggregate sample and for ensuring the production stability in the quarries and these are indispensable to guarantee the required quality and to ensure the checking of the quality level in case of ready constructed railway tracks. However, it's not suitable for modelling the railway loads (i.e. loads from vehicles and other effects) in a real way.

The authors worked out an individual laboratory test method (Fischer, 2015; Fischer, Németh, Harrach, Juhász, 2018), because other test methods can't consider the abrasion and breakage (real particle degradation) due to dynamic force and vibration. After the laboratory fatigue tests, the authors developed a special method with a CT (computer tomography, X-ray) equipment.

2. HISTORY OF THE RESEARCH

The research topic has prestigious international literature and sources in different topics as follows:

- laboratory tests;
- FEM modelling;
- DEM modelling;
- in-situ tests in railway tracks.

Foreign researchers dealt with different areas and worked out different methods, several special parameters, constants and indexes that helped the progression of the research:

- F_V (%);
- BBI;
- B_R;
- d<22.4 mm in mass percentage;
- d<0.5 mm in mass percentage;
- d<0.063 mm in mass percentage;
- d_{60}/d_{10} ratio;
- C_c ratio;
- M ratio;
- λ ratio.

The goal was to effort determine mathematical-physical trends and correlation between characteristics (see above point) and loading cycles of fatigue test.

The authors collected several significant results from the international literature review which can be read in the previous publication (Fischer, Németh, 2018a).

3. THE LABORATORY TESTS' PROCEDURE AND PARAMETERS

Basically the research is based on two kinds of research laboratory tests. The base of the first procedure is a special laboratory dynamic actuator. The second one is a simplified examination with CT equipment. These are described in the following chapters.

3.1. The laboratory test with the dynamic pulsator

This laboratory test method was developed as a part of an R&D financed by Colas Északkő Ltd. in 2014-2015. Thence several publications were written in this topic (Fischer, 2015; Fischer, 2017; Fischer, Németh, Harrach, Juhász, 2018; Fischer, Németh, 2018a; Fischer, Németh, 2018b; Juhász, Fischer, 2018; Juhász, Fischer, 2019a; Juhász, Fischer, 2019b; Juhász, Fischer, 2019c; Juhász, Fischer, 2019d).

In 2017 and 2018 the testing and evaluating method was accomplished by specify more precise deterioration process, considering only determined particle fraction, etc. The authors applied the following parameters during measurements and evaluation:

- two different types of railway ballast samples from andesite material and from different quarries;
- the samples are in accordance with MSZ EN 13450:2003 (MSZ EN 13450, 2003), A type, 31,5/50 mm, the authors received from Colas Északkő Ltd.;
- the samples have the following stone physic parameters (laboratory test were done by accredited laboratory of Colas Északkő Ltd.):
    - sample No. 1: LA_{RB} = 19 %, $M_{DE}RB$ = 17 %;
    - sample No. 2: LA_{RB} = 16 %, $M_{DE}RB$ = 4 %;
- dynamic tests with pulsator in different cycles (i.e. until 0.1, 0.2, 0.5, 1.0, 1.5, 3.0 and 5.0 million cycles), in every test with only fresh ballast material with particle fraction $d \geq 22.4$ mm (before pulsating $d < 22.4$ mm particles were screened out and they were not put back), where d is the size of the particle; in the 2014 series of measurements, the particle sizes below 22.4 mm were left in the tests, this was the reason why it is not possible to compare the current measurement results with the old ones;
- determination of PSD curves with screening related to sub-samples Before Pulsating (BP) test;
- determination of PSD curves with screening related to sub-samples After Pulsating (AP) test.

3.1.1. Presentation of the unique fatigue laboratory test

The individual laboratory testing method is a dynamic pulsating test for that the six lower frames of a 10-level steel shear box were used (Fischer, 2015). Frames were fixes together with steel metric screws. They prevent the horizontal relative displacements. The shear box contains some steel rolls which were not fixed to the down side of the bottom frame.

The assembly without the loading plate can be seen in Fig. 1.

Fig. 1: The sample of ballast material in place

The built-up layer structure is the following (see Tab. 1).

Tab. 1: The built-up layer structure

steel loading plate
D=300 mm steel plate with circular shape
(D=diameter)
0.46 x 0.42 m

<table>
<tr>
<td>30-cm-thick-layer
wooden sleepers
around the crushed stone samples</td>
<td>simple layer Viacon PP TC 1200 geotex.</td>
<td>30-cm-thick-layer
crushed stone
cross section 0.46 x 0.46 m</td>
<td>simple layer Viacon PP TC 1200 geotex.</td>
<td>30-cm-thick-layer
wooden sleepers
around the crushed stone samples</td>
</tr>
<tr>
<td colspan="5">simple layer
heat treated, non-woven, high strength geotextile with 1200 g/m² mass
type: Viacon GEO PP TC 1200
on the whole 1.0 x 1.0 m area</td>
</tr>
<tr>
<td colspan="5">10-cm-thick-layer
sand
type: E_2, 20.42 MPa according to MSZ 2509-3:1989
on the whole 1.0 x 1.0 m area</td>
</tr>
<tr>
<td colspan="5">simple layer
150g/m² mass geotextile
type: Naue Secutex 151 GRK
on the whole 1.0 x 1.0 m area</td>
</tr>
<tr>
<td colspan="5">20-cm-thick layer
eXtruded PolyStirol (XPS)
type: Austrotherm Thermoplan
sheets on the whole 1.0 x 1.0 m area</td>
</tr>
</table>

3.1.2. Research problems with the fatigue laboratory test

During the research there are several significant correlations between calculated parameters if the independent variable is the number of loading cycles (linear and power). The authors considered that it is a problem they did not achieve any results for five from all the ten parameters. The authors defined the time interval values of ballast screening based on technical prescriptions, standards and handbooks. This calculation could be executed for F_V, BBI and d<22.4 mm parameters. Only BBI gave nearly acceptable results, in case of the other parameters the results are not realistic, they can’t be accepted.

These results were published in previous publications (Fischer, 2017; Fischer, Németh, Harrach, Juhász, 2018; Fischer, Németh, 2018a; Fischer, Németh, 2018b; Juhász, Fischer, 2018; Juhász, Fischer, 2019a; Juhász, Fischer, 2019b; Juhász, Fischer, 2019c; Juhász, Fischer, 2019d).

The authors considered the following derelictions related to calculation of time intervals between ballast screenings:

- in the whole ballast cross section comparable amount of breakage is not formulated as the one that was measured in referred laboratory tests (e.g. there is hardly no breakage in the ballast shoulder, etc.);
- machine-made and/or manual tamping occurred breakage;
- only 225 kN axle load was taken into consideration (it is true for freight trains, for passenger trains about 180 kN value would be more realistic);

- other ballast polluting effects (e.g. dust, concrete sleeper abrasion, breakage, in case of water pockets the increase of fine particle content in the ballast bed because of evolving pumping effect due to repeated dynamic load, etc.);
- deterioration effect accelerated by substructure or superstructure defect;
- effects of other dynamic loadings (Kurhan, 2015; Kurhan, 2016);
- effects of track geometry and its degradation (Kurhan, 2015; Kurhan, 2016).

It is possible that these factors may over-realized the measurement method. That is an exercise to detect which are the most important factors for the research which we have to use during the tests.

The extruded polystyrol layer was significantly deformed during the dynamic test, so in 2019 the main goal is evolving a modified layer structure in that a stiffer and harder layer helps the laboratory test in better way. The laboratory test also a very time consuming process. It can take 1 to 1.5 months to achieve the 5 million cycles in the testing method. (In the authors' previous researches not only the maximum 5 million loading cycle value was considered, but the 'whole' deterioration process, i.e. the determination of PSD curves, as well as calculation of all the ten parameters were done after 100,000; 200,000; 500,000; 1 million; 1.5 million; 3 million and 5 million cycles for every ballast material samples with fresh (new) ballast for every measurement.)

3.2. Application of the CT (X-ray) equipment

While there is a wide range of literature available for the 3-D image analysis, the authors found very little source for testing with CT equipment (e.g. Fu, Hu, Zhou, 2017 related to granular materials, as well as Kozma, Halbritter, 2013; Kozma, Dorogi, Papp, 2014; Kozma, Zsoldos, Dorogi, Papp, 2014; Kozma, Zsoldos, Dorogi, Papp, 2015; Kozma, Zsoldos, Dorogi, Papp, 2016 related to other mechanical engineering examples). It is available to the authors in the laboratory of Audi Hungaria Faculty of Automotive Engineering at Széchenyi István University, Győr (Hungary).

Basic data of the device, and some relevant data for one measurement:

- A 360° rotation produces 1260 projections (CT-images).
- The number of the lines are 104.
- In case of multi slice: distance between two slices is 210 mm.
- Number of the pixels: 2048 × 2048 (used: 1024 × 1024).
- 2D-pixel size: 0.19124188 mm.
- 3D-XY-pixel size: 0.18966927 mm (the edge length of 1 spatial pixel – so called 'voxel').
- 3D-Z-pixel size: 0.1896692 mm.
- X-ray tube: Y.TU 450-D09.
- Tube voltage: 0…450 kV (used 210 kV).
- Current: 2.60 mA (it is related to 210 kV; e.g. 1.213 mA for 450 kV).
- Focus: small.
- Filter:
    - Al – 0.00 mm;
    - Cu – 1.50 mm;
    - Sn – 0.00 mm;
    - Pb – 0.00 mm.

The counted and 3-D scanned ballast particles are placed in a 140 mm diameter (inner dimension) HDPE tube with its original closing element (see Fig. 3-4) and put this tube into the CT equipment (digitally technic) (see Fig. 5). X-rays are a form of energy distribution in the family of electromagnetic vibrations. Computer tomography is a development of traditional X-ray screening technology.

Fig. 3-4: The HDPE tube with the sample

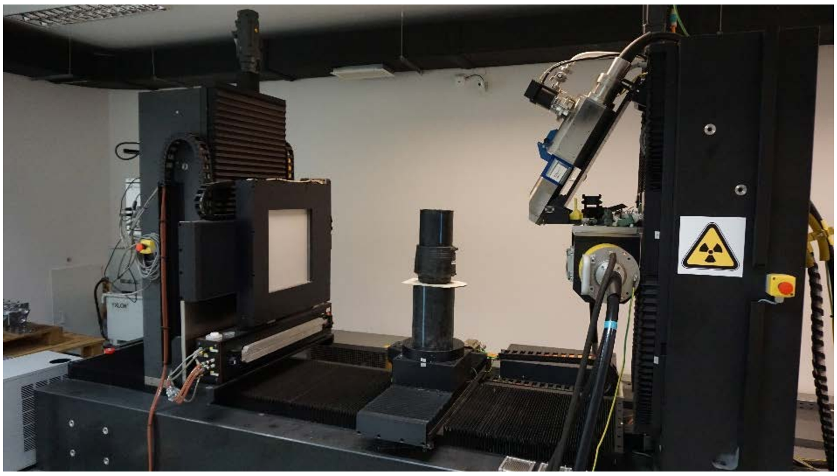

Fig. 5: The CT (X-ray) equipment with the HDPE tube

The measurement method is the following (for the 3-step loading):

- washing, drying and numbering of all the stones;
- measuring the weight of the stones and taking photo of all of them;
- inserting the stones into the HDPE tube (register the location of each particles: which raw, etc.);
- placing the HDPE tube into the CT equipment and recording CT 3-D model (initial model);
- loading with ZD-40 machine (until 300 kPa);
- another recording by CT equipment (2nd model);
- loading with ZD-40 machine (until 600 kPa);
- another recording by CT equipment (3rd model);
- loading with ZD-40 machine (until 900 kPa);
- another recording by CT equipment (4th model);
- measuring the particles after the loading (weight, photographing);
- washing and drying the particles;
- measuring the weight and photographing once again.

The one-step loading measurement method is very similar to the previous mentioned one, the authors had to save two recordings with the CT equipment (before as well as after loading) and the loading was up to 1800 kPa.

X-ray procedure

Homogeneous beam was emitted through the sample, which diffuses and penetrates (it depends on the material) and as a result of the adsorption, the distribution of the quantum of the X-ray changes and weakens in the image plane, blackens the detector to varying degrees. This creates the x-ray images; that depends on the quality of the ballast material. X-rays can also detect tiny cracks after the loading.

Computer tomography procedure

The object under examination is illuminated with a thin, flat X-ray beam. There is a detector which placed behind the object senses where and how much of the beam has been absorbed along a line. In the same plain, the beam is illuminated from several directions, and a drawing of the details in the plain (slice) is drawn from the measured intensity curves. The plain is then pushed away and rotated again. At the end of the procedure, the spatial structure of the test body can be mapped. "Structure" refers to the arrangement of details that can be distinguished from X-ray transmission capability. Modern CT (X-ray) equipment crawls several slices (up to 128) at a time, and a test can be performed in a few minutes with the necessary calculations. The available CT equipment can be seen in Fig. 5 and Fig. 6 shows an example for measurement.

Hopefully combined with 3D image analysis, the authors can take a new direction in our research.

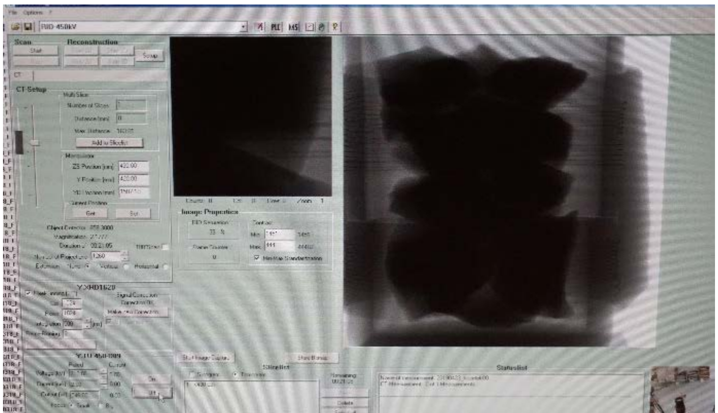

Fig. 6: Preview of a CT measurement, as well as the board of the software

Fig. 7-8 show images made by CT (X-ray) equipment and its original factory software. The samples in the HDPE tube were static loaded with compression stresses of given values (in Fig. 7. from left to right: 0, 300, 600 and 900 kPa; in Fig. 8. from left to right: 0, 1800 kPa).

The authors suppose that the speed of the loading is also a relevant parameter that will be examined in the future.

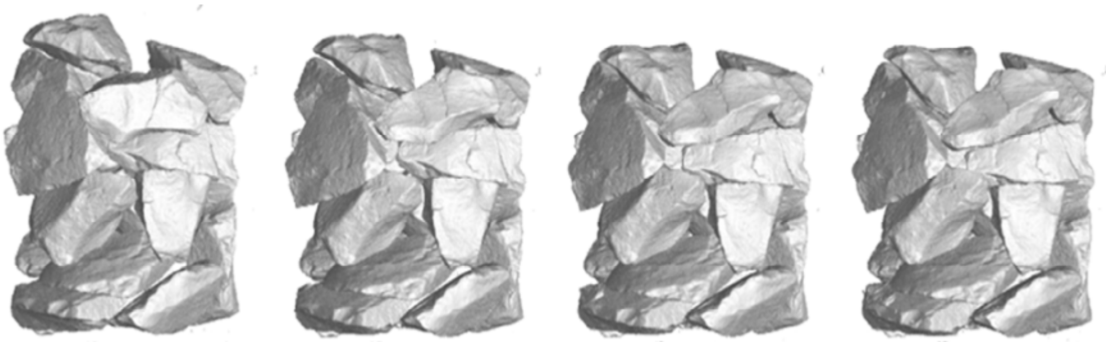

Fig. 7: CT images, 3-D models (samples in the HDPE tube with the different and increasing static loadings, i.e. from left to right: 0 kPa, 50 kPa, 200 kPa, 300 kPa) (own result)

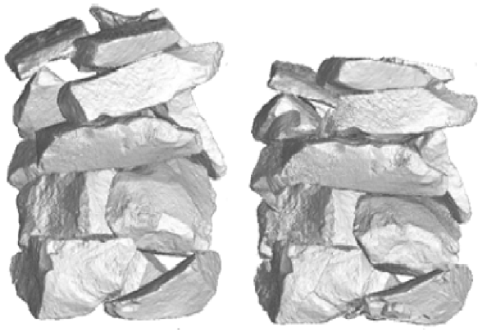

Fig. 8: CT images, 3-D models (samples in the HDPE tube with the different and increasing static loadings, i.e. from left to right: 0 kPa and 1800 kPa) (own result)

3.2.1. Findings and problems with the measuring of the CT equipment

At the second laboratory tests the evaluation of the results is still ongoing. The graphs below show the loading curve for the 3-step load and the one-step load (see Fig. 9 and 10).

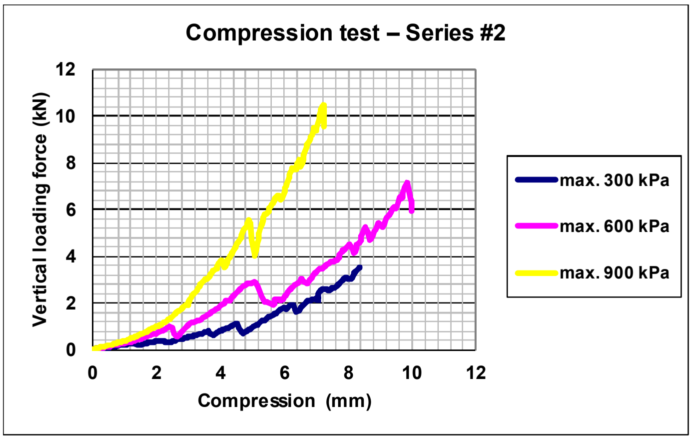

Fig. 9: Loading curves of loadings with max. 300-600-900 kPa compression stress values (3-step loading)

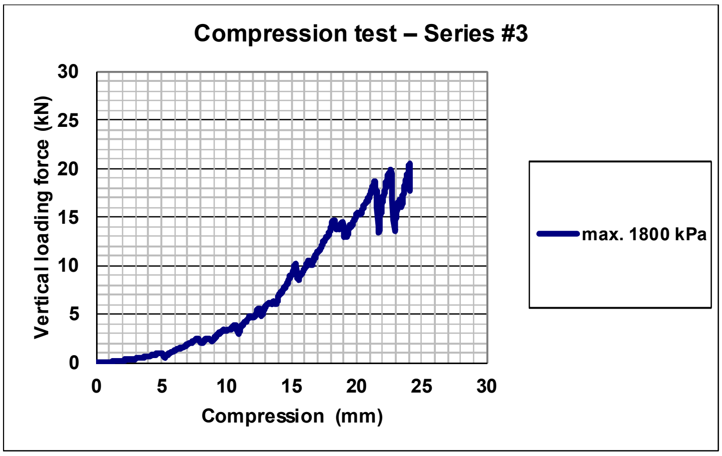

Fig. 10: Loading curve of loading with max. 1800 kPa compression stress value (one-step loading)

The loading curves show that they don't reach the upper limit of the load in a straight line, but they move down in certain places. In these places the ballast particles maybe break. With increasing the load, the fractures in the loading curve are increasingly greater. The loading curve rises much steeper after every (little) breakpoint.

The preliminary results show the quantity of the 'rest' particles from the entire ballast aggregate from the 3-step loading laboratory test (with the HDPE tube). This value calculated from the weight of the original numbered, washed and dried particles without loading and washed and dried particles after the latest loading (it is a percentage value, however it should be taken into account that up to 30% of some stone have broken down). The values can be seen in Tab. 1. The values written with slanted letters show the values that involve the rate of those stones that split into several pieces.

Tab. 1: The broken particles of the ballast aggregate after 300-600-900 kPa compression stress (the 3-step loading test)

Series number of the tests	The weight of the broken particles [%]
1	0.18%
2	0.14%
3	3.18% (6.26%)
4	1.13%
5	1.06% (3.90%)
6	1.65% (3.64%)

Tab. 1 shows that none of the samples contains more than 1.7% (with one expection – Series #3) of the powder of the stone after the latest loading. The percentage of broken stones is also less than 4%, including the number of stones that fell into several larger pieces (not powder, like the previous case).

The authors also investigated that in the tube which particles were broken down mostly of the rows. Tab. 2 shows the degradation for each rows (ratio of after-loading-weight and before-loading-weight in percentage).

Tab. 2: The broken particles of the rows at the 3-step loading test

Series number of the tests	Number of the row	The weight of the broken particles [%]
1	1	0.10%
	2	0.06%
	3	0.13%
	4	0.13%
	5	**0.47%**
2	1	0.00%
	2	0.27%
	3	0.11%
	4	0.09%
	5	**0.27%**
	6	0.09%
3	1	5.45%
	2	0.62%
	3	1.28%
	4	**5.73%**
	5	2.80%
4	1	0.15%
	2	1.59%
	3	**2.04%**
	4	0.57%
	5	1.17%
5	1	0.24%
	2	0.34%
	3	0.67%
	4	0.31%
	5	**4.33%**
	6	0.21%
6	1	0.51%
	2	0.05%
	3	0.54%
	4	0.35%
	5	**6.68%**
	6	0.43%
	7	0.91%

According to the values the particles are mostly broken in the upper and middle part of the aggregate (row #1 is the lowest 'plane' in Tab. 2).

During the test particles in the HDPE tube moved significantly during the test (mainly on top of the aggregate). The authors would like to analyse particle movements in the future.

4. ORIGINALITY AND PRACTICAL VALUE

The most important goal of the authors is to supplement the currently used regulation with new measurement methods, because the original standardized tests don't consider the realistic loading of ballast samples. The authors' developed and new methods may serve as a basis for a future instruction or regulation.

5. CONCLUSIONS

The authors would like to reduce the time requirement of newly developed testing methods with improved manner. The authors combine the compression tests with 3-D image analysis (fulfilled 3-D shape measurement) with the help of CT (X-ray) equipment. The measurement method was developed; the procedure of evaluation methodology is in progress.

Beside them field tests are planned in the Hungarian railway lines. The authors plan to collect samples from old railway lines where ballast aggregates have known PSD (particle size distribution) at the time of construction.

In the laboratory the authors always work in idealized conditions. This is the reason why the particle breakage values are much higher than the values in real circumstances (see measurement results from 2014 and 2017-2018). Besides, the authors could test only one kind of loadings. Tamping machines also break ballast particles during work, so this kind of effect is also needed to be considered in the future research. Delivery of the crushed stone to the site can also be an important parameter that has to be considered.

The authors plan to work with DEM simulations (Fu, Hu, Zhou, 2017; Orosz, Tamás, Rádics, Zwierczyk, 2018; Orosz, Tamás, Rádics, 2017a; Orosz, Tamás, Rádics, 2017b), for this a spatial model must be built. The simulations with the laboratory tests would be comparable.

6. ACKNOWLEDGEMENTS

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GOTTHARD: FROM THE PATH TO THE RAILWAY BASE TUNNEL - 2000 YEARS OF THE ALPINE CROSSING

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SUMMARY

The European North-South connection goes through the Alps. Gotthard is the most important crossing in Switzerland. As found archaeological objects (counter) prove that also the Romans used this axis. In the 16th century, 1.500 tons of goods crossed the Gotthard annually. Due to the flourishing and development of freight traffic and tourism between the 19th and 21st century, it was always necessary to provide greater "Gotthard capacity" for passenger and freight traffic. The presentation describes the major "milestones" of these developments.

1. FROM THE PATH TO THE STAGE COACH "MAN-MADE ROAD" 1831

The first Alpine crossings were paths beaten by people and animals. The width of 2-3 m was determined by the crossing of two packed mules.

Fig. 1: Mules and drovers

Later, larger stones from brooks were laid on the roads as pavement. The Gotthard Pass lies 2106 meters above sea level. Thus far, in winter 5-10 m high snow cover the road, which means that even nowadays the mountain pass is usually closed from Christmas to Pentecost! In the 15th century, couriers were already regularly crossing the pass. From the 17th century onwards, horse couriers carried the mail across the Alps.

Saumkolonne beim Urnerloch, 1790.

Fig. 2: Path with a tunnel

Freight traffic also increased rapidly; in the 16th century, 1-200 tons of goods were transported each year through the Swiss Alps! Already in the 19th century this number had increased tenfold to 2-3000 tons per year. In comparison: in 2016 20.1 million tonnes were transported on rail, 10.7 million tonnes were transported by trucks through the Swiss Alps.

There was not only freight and passenger transportation over the Gotthard, there were also war actions on this crossing. The carthaginian general Hannibal from North Africa wanted to surprise the Romans from the north. In those times - 231 BC - , he accomplished an incredible journey: On the northern coast of Africa he cut across to Gibraltar, from there to the Iberian Peninsula, through the Pyrenees, in France through the Rhone Valley and across the Gotthard Pass.

Fig. 3: Hannibal on Gotthard

And all of this with 50.000 soldiers, 9.000 horsemen and 37 fight-elephants! The other major battle at Gotthard was fought by the Russian General Suvorow in 1799 against Napoleon's troops. Out of Suvorow's 26.000 soldiers 7.000 fell at the "devil's bridge" on the north side of the pass. Suvorow crossed seven passes in 20 days! He virtually passed through all of Switzerland and, of course, robbed everything. Below the pass, on the north side, is a Russian war memorial reminding of this battle.

From the beginning of the 19^{th} century, due to the flourishing of freight traffic and tourism, the pass became increasingly important. Thus, both - the northern and southern cantons - began building the Gotthard route in 1825. As early as 1830, the journey to from Basel to Milano (300 km) lasted 4-5 days. From 1835 on already three mailcarts a week crossed the Gotthard pass. From 1840 onwards, a horsecart for 10 people drawn by ten horses ran over the new man-made road.

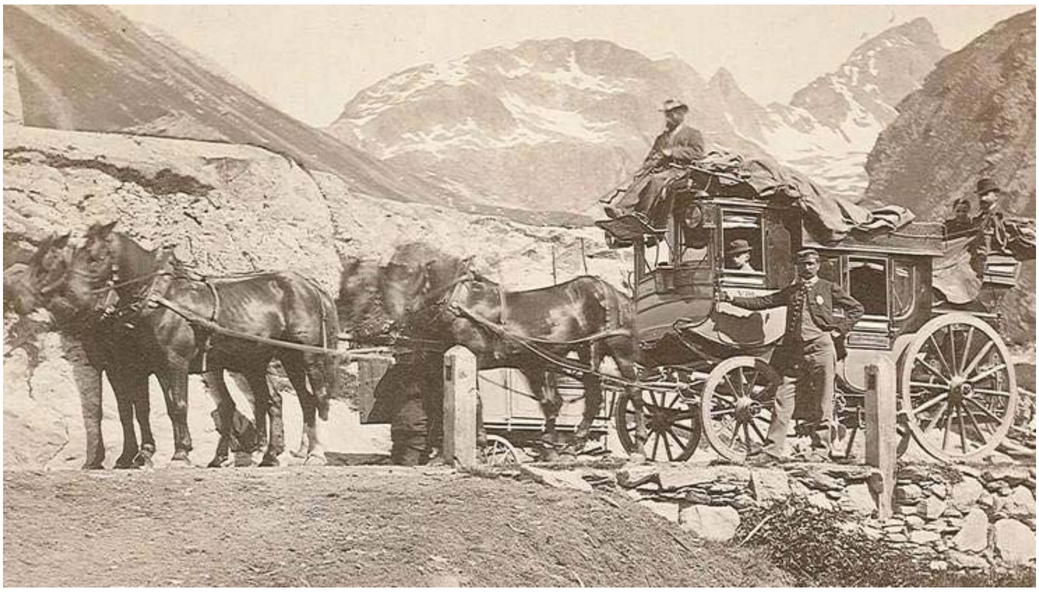

Fig. 4: Stage coach

This stage coach-route took 50 hours from Basel to Milano! The construction of the road in the mountains, above the gorges and through the valleys required many civil engineering facilities. Many rubble viaducts were built and several tunnels had to be cut, of course only where the geological conditions were so good that the cavity was self righting! On the south side of the pass - on the Tremolan - the mountain is so steep that only with 23 hairpin bends the height difference could be bridged. Of course, the construction, maintenance (snow removal, etc.) and operation of the road provided a good income opportunity for poor mountain peasants.

With the pass-crossing a whole new industry evolved: cattle riders, mule riders, harness-makers, coachsmiths, rope makers, feed manufacturers, and of course the entire hotel- and catering industry (hoteliers, restaurateurs) and so on.

2. THE FIRST RAILWAY 1872-1882

The first ideas arose already in 1851. But it was not until 1871 that the Gotthard Railway Company was founded. Only after one year, the Gotthard tunnel project was tendered and a contractor was chosen to build the tunnel. Alfred Escher - a politician from

Zürich - was the main engine of the new railway line. The new line was 152 km long, including 228 bridges and all together 30 km in tunnels. The Pass Tunnel with its 14.984 m is the longest build-up below the ground. To get over the height differences, several turntunnels (corkscrew tunnels) had to be built. The highest point of the Pass Tunnel is situated at 1131 m above sea level. The Pass Tunnel was broken by 3800 workers from both sides. Unfortunately there were 180 fatal accidents. Other hundred Italian workers died because of a stomachworm, that developed due to bad hygiene conditions, or they died later with "fine stone dust lungs" (silicosis).

The needed holes for the blasting were already drilled with a six-arms-pneumatic drill car. The explosions were already partially done by using dynamite. The broken rock was pushed out of the tunnel by hand on trams, later by wagons, that were pulled by compressed air and steam-powered locomotives. The breakthrough took place on 28th of February in 1880 with great accuracy. The geodetic error (after approx. 2x8 km of excavation) was only 33 cm horizontally and 5 cm vertically. The average working speed was 4.4m/day. As a matter of course the tunnel was built in 3x8 hours shifts. The maximal overburden was 1900m, because of that the air temperature in the tunnel rose to 30-40 degrees Celsius. In several zones with high rockpressure, the thickness of the walling - after repeated rebuilding and reconstruction - reached 3 m.

In May 1882, with a three-day celebration, the tunnel was inaugurated and put into operation. The tunnel itself cost 66 million Frank (1882). These days, black flags fluttered in the Canton Uri, because the population thought that the industry that had developed over the past 50-100 years, would be unnecessary due to the new railway and many citizens would be left without work. But that didn't happen, because the railway needed a lot of manpower due to the high volume of passenger and freight traffic. Already in 1883 one million passengers were transported by the Gotthard railway. By 1888, 32 trains had crossed the tunnel daily. In the 1920s and 1940s, already electric locomotives were on the line.

3. THE MOTORWAY OVER THE GOTTHARD 1969-1980

In the 1950s and 1960s, the number of automobiles and trucks and their traffic in Switzerland increased very rapidly. The building of the Swiss motorway network was started. The Gotthard Pass Tunnel is one of the longest tunnels in Europe, with a length of 16.942 m. The tunnel - because of the high cost - has only two lanes, not like the access highway which has 2x2 lanes. Next to the main tunnel is another emergency tunnel. Thanks to this tunnel, the northern, German-speaking Switzerland has a "winter-safe" and virtually rock-free connection with the southern, Italian-speaking Switzerland.

The mostly Italian 500 workers have been working on this tunnel for 11 years. The ventilation of a nearly 17 km car tunnel is only possible with full cross-flow ventilation. Accordingly, four ventilation shafts (hundreds of meters deep) are required. Above the traffic clearance there are airducts for fresh and polluted air. The cross-sections of the tunnel, depending on the length of the 'ventilation section', are therefore 69-96 m^2. The average digging speed was 12 m per day. Due to the high overburden, the air temperature exceeded 35 degrees Celsius. During the 11-year building time, 19 fatalities occurred. The main tunnel alone cost 686 million Frank (1980). The tunnel is very dangerous due to the counter traffic. Unfortunately, between 1980 and 2018, 40 people were killed in accidents. On the other hand, during heavy traffic days: Easter, Pentecost, Christmas and summer holidays, there are many congestions at the end of the four lane highway and in front of the two lane tunnels, which means a 3-4 hour wait! Of course, tracing through the mountains and valleys required a lot of tunnels and bridges, which of course caused high construction costs. The Gotthard motorway is one of the most expensive lines in the Swiss motorway network.

4. THE GOTTHARD RAILWAY BASE TUNNEL 1996-2016

The 57 km base tunnel is the longest tunnel in the world. The highest point of the tunnel is situated only 571m above sea level. Above the tunnel is a 2 to 2500m high overburden. The tunnel consists of two single track pipes. The two pipes are connected by a cross-over every 325 m, so in case of an accident it is possible to pass from one pipe to the other. In addition, emergency stations and escape systems are located at tertiary points. The Swiss State Railway chose this tunnel system for operational and maintenance reasons. Passenger trains run at 250 km/h and freight trains run at 160 km/h. In terms of speed, the smallest radius of curvature is 5000 m and the maximum allowed climb is only 12.5 ‰. On the first railway axis (1872-82), due to the high elevations, a second, and sometimes a third, locomotive had to be connected to the freight train assembly in order to be able to lift heavy weight on the mountain. Of course, this is a big loss of time. With the new tracing this is no longer necessary, and it even fits the new freight trains (1500 m long and 4000 tons weight).

Fig. 5: The Gotthard tunnels: 1882: railway; 1980: motorway; 2016: base tunnel for railway

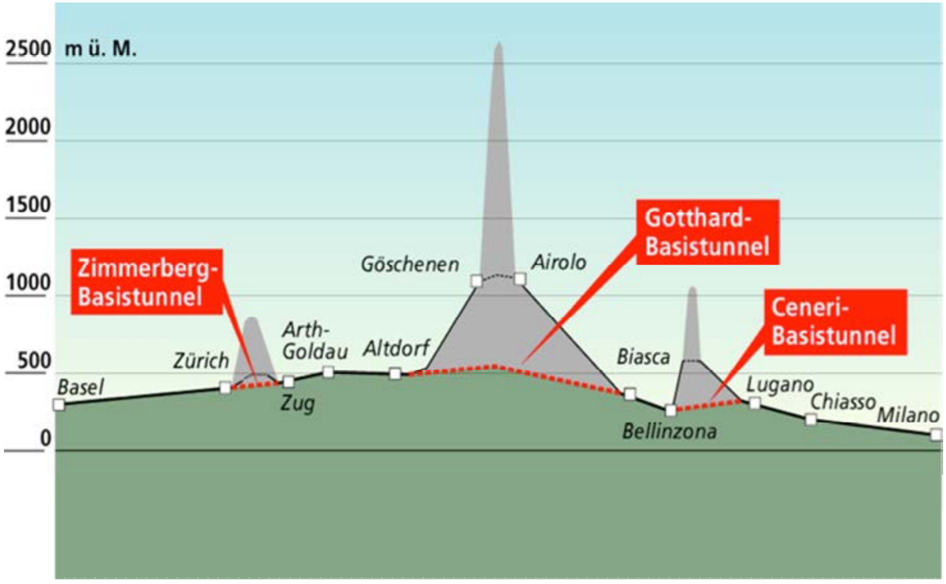

Fig. 6: Longitudinal section: railway 1, railway 2

Why did Switzerland build this very long and deep tunnel? Especially for environmental reasons! Switzerland wants to shift passenger and freight traffic from roads to rails! But this requires proper opportunities. Already today, Switzerland accounts for two thirds (20.1 million tonnes) of its annual freight traffic on railway and 1/3 (10.7 million tonnes) of freight on the roads. In Austria and France the ratio is just the opposite!

Special key problems of the base tunnel, which required unusual solutions.

- *The large length:* intermediate opening points are needed for the building to reduce construction time. These were solved with three access tunnels and an 800m deep shaft. All logistics - during construction - must be carefully and predictably planned. Possible accidents and fires in the plant require special solutions for rescue.
- *The large overburden* (2'000-2'500m): high mountain pressures occur. In the case of hard rock, this is the so-called *true/real* mountain pressure, which results in a sudden peeling of large rock masses (1-5 m3 large). If the tunnel is in soft rock, the *true/real* mountain pressure causes a large deformation of 60-80 cm convergence. In such rock cover, the rock temperature is 40-50 degrees Celsius. In such cases, the air in the working area had to be cooled to 28 degrees Celsius and properly ventilated. Since we wanted to develop most of the tunnels with a Tunnel Boring Machine (TBM), the main question was whether the cutting head of the machine would get stuck due to deformation? Except for certain sections (for example Sedrun), 80% (91 km) of tunnels were broken with the TBM. The tunnels were built with four such TBMs. Unfortunately, there were 9 fatal accidents.

The cost of the tunnel amounted to 12.2 billion Franks (2017).

Acquired experiences: over twenty-five years of planning and implementing a lot of interesting and valuable experience has been gained. The following are the most important:

- the special conditions demanded several pioneering achievements;
- these required, in part, technical and political courage;
- with the two new axis (Gotthard and Lötschberg), the freight capacity of freight trains across the Swiss Alps has tripled;

- the four TBMs (ø = 9.58 m) also proved to be successful, despite high compressive strength, abrasion and convergence averaging 10-15 m per day. The maximum quarry was 56 m in a day with a TBM;
- from the intermediate opening points, tunnels can be driven in five places at a time
- of course, there were smaller, bigger problems with such a "century project". But in retrospect, implementation can be said to have been successful, which means that the costs and execution time were within the projected frames.
- the largest geodetic errors at breakouts of various tunnels was 13.7 cm horizontally and 1.7 cm vertically by a total of 17 km for "layout lengths"
- endangering of dams (this is a special non-routine problem):
  - There are three dams along (800 m) the axis of the tunnels and above (1500 m). Based on a number of previous bad experiences, the client knew that the risk here is high and there could be millions in damages. When draining or tapping water from the hollows and pits of the mountains, the empty hollows and gaps are slowly closed due to the high mountain pressure. This deformation slowly "spreads" to the surface and there lowerings may occur. These lowerings can cause damage on the dam. The dams are in valleys and they cannot be reached in winter (a lot of snow and foggy days). Because of this, on the one hand a "winter-proof" and totally independent - world first of its kind - monitoring-system had to be installed.
  - On the other hand, it has already been decided in advance that in case of waterbreak-in what kind of waterproofing works will be done and what materials will be used to reduce the amount of water. After a major waterbreak-in, it was verified that the monitoring and all the planned “insulation works” were successful. The top of the dam sank -60 mm and the bottom of the lake sank -100 mm. Another interesting phenomenon has been identified: the valley "breathes". That means that in summer the valley will be 40-50 mm narrower due to temperature and water pressure and vice versa! The dams were not damaged and the electricity works were able to operate normally .

5. CONCLUSIONS

This 2000 year evolution is reflected in the following figures:

- Initially, 3 mules crossed the pass with a load of 80-200 kg per day
- Daily traffic today: in 2017 already 18.000 cars went through the Gotthard axis. Through the old rail- and base tunnels run 400 freight trains with a burden of 55-60.000 tons and in addition 80-120 passengertrains daily.

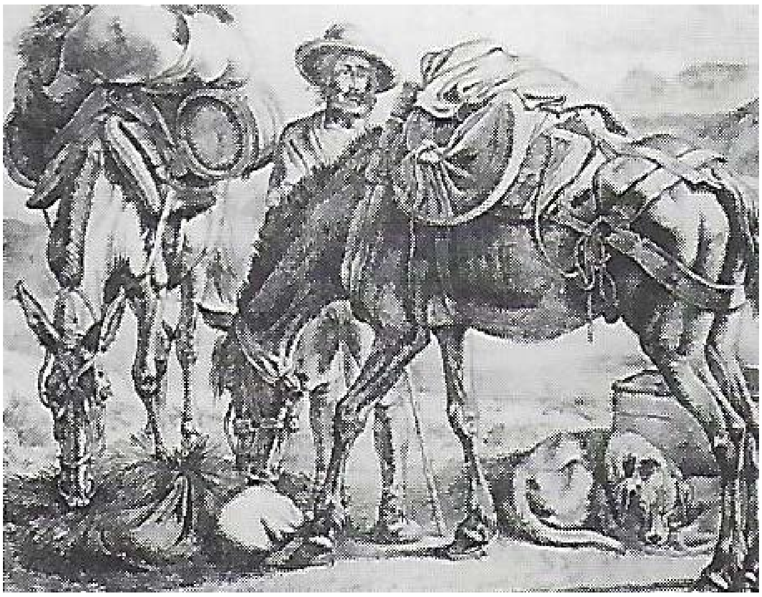

Fig. 7: Mule with drover

Fig. 8: Train today (v=250-300 km/h)

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NEW MATERIALS FOR CONCRETE BRIDGES

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SUMMARY

Development of new types of concretes and new types of reinforcements provide a wide range of possibilities for improving the characteristics of bridges.

Development of materiel characteristics, like high strength concrete, high performance concrete, lightweight concrete, non-metallic reinforcements etc. are advanced solutions.

Intention was herein to give international examples of efficient solutions in addition to recent research results.

1. INTRODUCTION

Present paper intends to review major developments of concrete bridges.

These include development of new types of concretes and new types of reinforcements in addition to increase of spans or durability or just an interesting engineering solution.

Development of new types if concretes include: high strength high performance concrete bridges, lightweight aggregate concrete bridges, high strength, non-corrosive fibre reinforced polymer reinforcement in bridges.

Details are given on three running projects supported by Hungary research grants.

2. HIGH STRENGTH HIGH PERFORMANCE PEDESTRIAN BRIDGES

2.1. UHPC bridge, Sherbrooke, Canada

The first large span (60 m) RPC (Reactive Powder Concrete) or UHPC (Ultra High Performance Concrete) pedestrian bridge has been erected in Sherbrooke in 1997, Canada (Fig. 1 and 2). It is called as Passerelle of Sherbrooke. It consists of 6 pieces of 10 m long match-cast elements with two post tensioned bottom arches and post tensioned inclined diagonals. The structure has been completed by external post tensioning (Fig. 1 and 2) (Aïtcin, 1998).

Selection of materials was done by the University of Sherbrooke under the supervision of Prof. Pierre-Claude Aïtcin. The deck is 30 mm thick and post tensioned longitudinally as well as transversally. Post tensioned diagonals connect the deck to the two bottom arches made of stainless steel tubes of 2 mm thickness and 3.2 m length filled with RPC. The RPC contained relatively high amount of modified CEM II type cement having low hydration heat, silica fume in addition to crushed quartz, sand, superplasticizer and water with low water-cement ratio. The elements were steam cured. The RPC reached an average strength of 199 MPa with a standard deviation of 9.5 MPa, the modulus of elasticity was 48,000 MPa and the modulus of rupture was 40 MPa (Aïtcin, 2014).

Fig. 1: View of Passarelle of Sherbrooke, Canada (Photo by Balázs)

Fig. 2: Longitudinal post tensioning of Passarelle of Sherbrooke, Canada (Photo by Balázs)

2.2. Seonyugyo Bridge, Seoul, South-Korea

A 120 m span UHPFRC pedestrian bridge (called: Seonyugyo Bridge or Rainbow Bridge) has been erected in Seoul, South-Korea on the occasion of the 100 years of diplomatic relations between Korea and France designed by Rudy Ricciotti (Fig. 3). The main arch is made of Ductal® containing high percentage of steel fibres.

Fig. 3: Seonyugyo Bridge: 120 m span UHPC pedestrian bridge in Seoul, South Korea (photo by Balázs)

2.3. MuCEM footbridge, Marseille, France

The MuCEM footbridge has a particular role to connect the Museum of European and Mediterranean Civilizations (MuCEM) to the Fort Saint-Jean in Marseille (Fig. 4). It is a very elegant solution to bridge the gap between the two architectural styles and constructions with a highly elevated structure. The MuCEM footbridge is constructed of precast segments of Ductal® each of 4.60 m, created from a single mould and including high steel fibre dosages. The precast elements are post tensioned together.

The French Association of Civil Engineering (AFGC) developed recommendations for design of UHPFRC structural elements (AFGC, 2007; AFGC, 2013).

Fig. 4: UHPFRC pedestrian bridge between the Museum of European and Mediterranean Civilizations (MuCEM) to the Fort Saint-Jean in Marseille (photo by Balázs) (Balázs, Farkas, Kovács, 2016)

2.4. Shell pedestrian bridge in Madrid

Two pedestrian bridges (Matadero and Invenadero Bridges) with cover of a concrete shell has been constructed in Madrid on the banks of Manzanares River (Corres, Diestre, León, Pérez, Sánches, Cruz, 2012) (Fig. 5). These pedestrian bridges are excellent examples of creativity, optimal use of material and intention for extraordinary appearance. Purpose of these special bridges was to establish communication between downtown Madrid and its surroundings. The structural solution consists of a reinforced concrete arch-vault with suspended composite deck spanning 43.5 m and 7.7 m rise. The deck is suspended by means of two series of 8.1 mm diameter ties every 0.6 m at both sides.

Fig. 5: Shell pedestrian bridge in Madrid (Photo by Balázs)

3. LARGE SPAN BRIDGES

3.1. Millau viaduct, France

The Millau viaduct in France (Fig. 6) is one of the most spectacular cable stayed viaduct.

As we know it from Michel Virlogeux, the structural designer of Millau viaduct, its construction took 17 years from the idea to the realization.

It is cable stayed viaduct with the deck situated 270 m above the level of the small river Tarn.

It become a landmark soon with high appreciation both of engineers as well as non-engineers.

Fig. 6: Millau viaduct, France (Photo by Balázs)

3.2. Térénez Bridge, Brittany, France

The Térénez Bridge, Brittany, France is a very spectacular cable stayed bridge constructed in 2011 (Fig. 7). Overall length is L=525 m, span is l=285 m, it has a horizontal curvature r=800 m. The most impressive are the lambda pylons. It has been awarded as winner in 2014 for *fib* Awards for Outstanding Concrete Structures.

Fig. 7: Térénez Bridge, Brittany, France (fib Awards for Outstanding Concrete Structures, Winner, 2014)

3.3. Hoover Dam Bypass Bridge, Large span arch bridge, Colorado, USA

With a main span of 323 meters, the Hoover Dam Bypass Bridge (called also: Mike O'Callaghan – Pat Tillman Memorial Bridge) is the fourth-longest, single-span concrete arch bridge in the world (Fig. 8). Each half arch rib is made up of 26 cast-in-place sections, with construction starting from the canyon walls and a closure pour that locks the two halves together. Approximately 6,880 m^3 of concrete of 69 MPa strength is cast in the arches. The outer dimensions of each hollow arch rib are 6 m wide by 4.26 m. Structural steel struts connect the arches at each column and are covered with precast concrete panels. The largest struts weigh nearly 40 tonnes. The 3 m-tall concrete segments were each precast offsite and erected to form the pier columns. The precast columns are 90 m tall. The structural steel tub girders were fabricated offsite and placed with cableway cranes. The temporary cable stay tower and support system for erection of the arch incorporated more than 600,000 m of cable-stayed strand. The bridge design satisfies objectives for both architecture and performance.

Fig. 8: Hoover Dam Bridge, photo is taken from the Hoover Dam (Photo by Balázs)

4. EXAMPLES FORM NEW DEVELOPMENTS FROM JAPAN

4.1. Tomai Expressway, Shizuoka, Japan

The new Tomai Expressway between Tokyo and Kyoto is constructed parallel to the first Tokai Expressway in order to avoid congestion of traffic. The Tomai Expressway is the most heavily used road operated by the Central Nippon Expressway. In some section with more than 100,000 vehicles a day.

Earthquake resistance has been one of the main aspect for design of the freeway viaduct in the vicinity of Shizuoka. A special solution has been developed for the web. The web is consisted of steel tubes cast with concrete providing not only reduced weight but also transparency for the superstructure (Fig. 9).

Fig. 9: Viaduct on the new Tomei freeway between Tokyo and Kyoto at Shizuoka (photo by Balázs) (Balázs, Farkas, Kovács, 2016)

4.2. Butterfly web bridge, Terasako Choucho Bridge, Japan

Butterfly web bridge is named after the shape of the prefabricated web of 150 mm thickness (Fig. 10). The web is prestressed along one of the diagonals which is subjected to tension with 15.2 mm diameter strands of indented surface. The 80 MPa design strength concrete includes short steel fibres. The web does not contain non-prestressed reinforcement.

One of the reasons for this special web is the intention to reduce weight of the structure, hence increase earthquake resistance. The bridge is constructed by using the balanced cantilever method. By using butterfly webs, not only the earthquake resistance but also sustainability improves because less concrete is required compared to an ordinary concrete web box girder.

Fig. 10: Side view of Terasako Choucho Bridge with butterfly web, Miyazaki, Japan (Courtesy of Akio Kasuga, Sumitomo Mitsui Construction, Japan) (Balázs, Farkas, Kovács, 2016)

Terasako Choucho Bridge has been constructed with butterfly webs. It received the Tanaka Award of Japan Society of Civil Engineers in 2013 (Fig. 11). It is a 10-span continuous butterfly web bridge with a length of 712.5 m, spans of 58.6 m + 87.5 m + 7×73.5 m + 49.2 m and width of 9.26 m.

Fig. 11: Prestressing of main girder by the Terasako Choucho Bridge with butterfly web, Miyazaki, Japan (Courtesy of Akio Kasuga, Sumitomo Mitsui Construction, Japan)

5. LIGHTWEIGHT CONCRETE BRIDGES

5.1. Lightweight concrete bridge, Stolma Bridge, Norway

The Stolma bridge in Norway had the longest span of lightweight aggregate concrete bridges in 2000 (Fig. 12) with the main span of 301 m (total length 467 m). Concrete grade was LC 60 with density of 1930 kg/m^3.

Fig. 12: Lightweight aggregate concrete bridge (Stolma bridge) under construction in Norway (fib bulletin 8)

5.2. Recent research project with lightweight concrete bridges

We have a research project grant VEKOP-2.1.1-15 on „*Development of high performance bridge girder families of normal weight or lightweight aggregates with inclined tendons*" which is a continuation of research grant GOP KMR Nr. 12-1_2012-0156 „*Development of lightweight concrete for infrastructure*". We are reporting here the preliminary results on lightweight concrete bridge girder.

5.2.1. Pilot test for lightweight concrete bridge girder (1st girder in Hungary)

Bridge girder has been cast by Liapor lightweight aggregates and lightweight aggregate concrete are presented in Fig. 13. It has been successfully used for preparing a lightweight concrete prestressed pretensioned concrete girder. Its cross-section is shown in Fig. 14. Fig. 14. gives the demonstration of bond between the prestressed tendon and the lightweight concrete are presented.

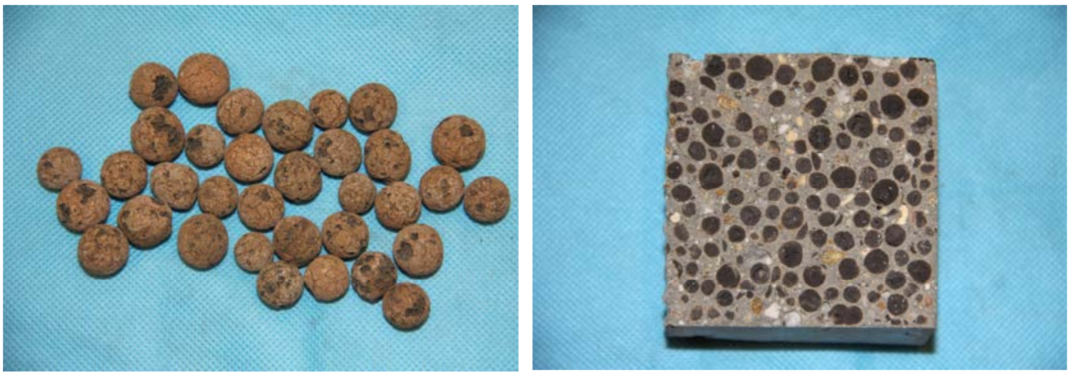


Fig. 13: Lightweight aggregates and lightweight aggregate concrete

Fig. 14: Precast lightweight bridge girder and demonstration of bond

5.2.2. MAÚT e-ÚT 07.01.21 (2016) for resistance against chloride migration

We developed an important specification for Design Guide by MAÚT e-ÚT 07.01.21 (2016) „*Design and Production of Precast Lightweight Concrete Structural Elements for Infrastructure*" by defining requirements for minimum concrete cover as a function of chloride migration coefficient at 28 days: $D_{\mathrm{RCM},28}$ for environmental classes XD and XS based on test NT Build 492 according to RCM (Rapid Chloride Migration) (Fig. 15). Values in italic in Tab. 1 are proposed values.

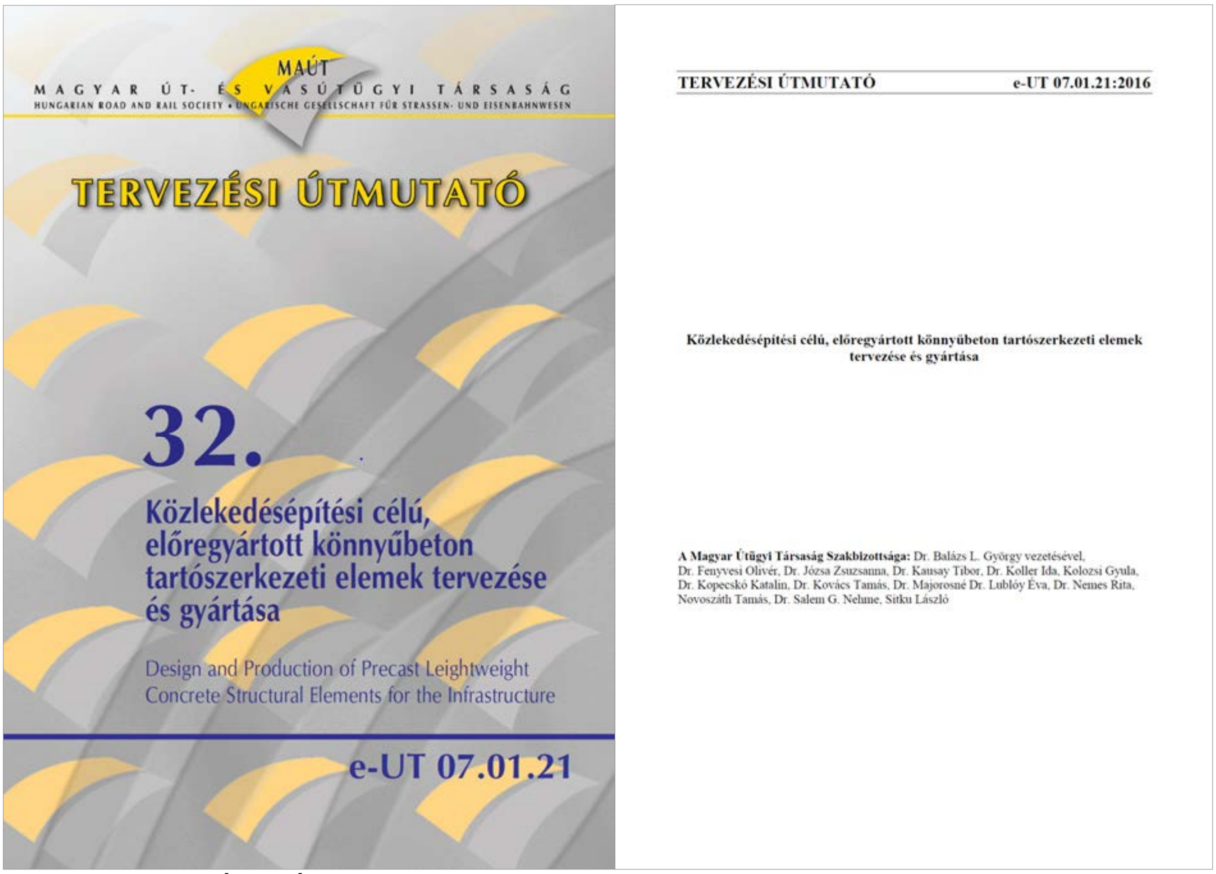


Fig. 15: MAÚT e-ÚT 07.01.21 (2016) „Design and Production of Precast Lightweight Concrete Structural Elements for Infrastructure" Design Guide

Tab. 1: Max. values of $D_{RCM,28}$ by considering 100 years design life (after Wegen, Polder, Breugel, 2012 and Polder, Wegen, Breugel, 2010)

Concrete cover [mm]		*Proposed max.* $D_{RCM,28}$ values [10^{-12} m^2/s]							
		CEM I		CEM I + III 25-50 m% Slag (S)		CEM III 50-80 m% Slag (S)		CEM II/B-V, CEM I + 20-30 m% Fly ash (V)	
Rein-forcing bar	Pre-stress-ing steel	XD1 XD2 XD3 XS1	XS2 XS3	XD1 XD2 XD3 XS1	XS2 XS3	XD1 XD2 XD3 XS1	XS2 XS3	XD1 XD2 XD3 XS1	XS2 XS3
35	45	*3.0*	*1.5*	*2.0*	*1.0*	*2.0*	*1.0*	*6.5*	*5.5*
40	50	5.5	*2.0*	4.0	*1.5*	4.0	*1.5*	12.0	10.0
45	55	8.5	*3.5*	6.0	*2.5*	6.0	2.5	18.0	15.0
50	60	12.0	5.0	9.0	*3.5*	8.5	3.5	26.0	22.0
55	65	17.0	7.0	*12.0*	5.0	*12.0*	5.0	*36.0*	*30.0*
60	70	22.0	9.0	*16.0*	6.5	*15.0*	6.5	*47.0*	*39.0*

6. DURABILITY ASPECTS – INCREASED FREEZE-THAW RESISTANCE

By the recent research project supported by the Hungarian Research Grant NVKP_16-1-2016-0019 entitled "*Development of concrete products with improved resistance to chemical corrosion, fire or freeze-thaw*" one of the major aspects is to increase durability of bridges by improved freeze-thaw resistance.

It has been already internationally recognized that durability of concrete is improved by applying air entrainment admixture. Efficiency of air entrainment agent depends on the factors like: type and amount of air entrainment admixture, composition and consistency of concrete, cement type, type of mixing and ambient temperature (Kausay, 2008).

Scanning Electron Microscopy is an efficient tool to analyse the internal structure of concrete (Kopecskó, Balázs, 2019). Air entrained pores are recognizable in Fig. 16. Efficient pores for durability have diameters from 20 to 300 microns. In Fig. 17 ettringite, portlandite and micro cracks are present in an entrained air pore. In Fig. 18 a microcrack and two carbonated pores are observed.

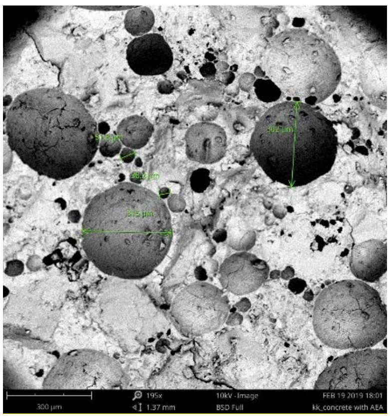


Fig. 16: Entrained air in concrete: efficient size of pores: 20-300 micron, magnification: 195×

Fig. 17: Ettringite and portlandite, microcracks in an entrained air pores, magnification: 790×

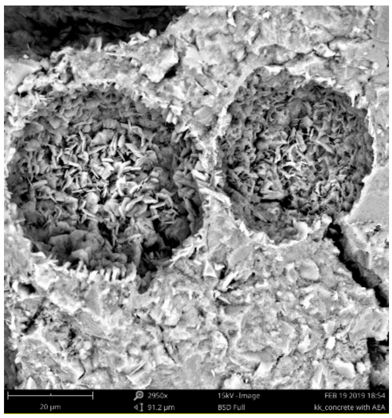


Fig. 18: Microcrack through a pore and pores after carbonation, magnification: 2950×

7. NON-METELLIC REINFORCEMENTS FOR INCREASING DURABILITY OF BRIDGES

In this Chapter we intend to report our results or intentions in applying high strength, non-metallic reinforcements as internal embedded reinforcement or externally bonded reinforcement.

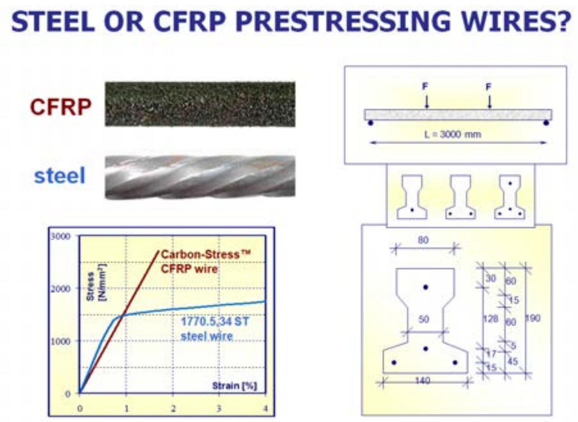
At the moment we are supported with grant from the Hungarian Ministry of Innovation 2018-1.3.1-VKE especially for the applications of high strength, non-corrosive fibre reinforced polymer reinforcement in bridges.

In addition to the projects mentioned in the following, we participated in two European Marie Curie Thematic Network Projects: ENCORE NRTN: CT-2004-512397 Projects: "*European Network for Composite Reinforcement*" as well as the ENDURE: PITN-GA_2013-607851: "*European Network for Durable Reinforcement and Rehabilitation Solutions*" (reported: Szabó, Balázs, 2011; Bilotta et al, 2015; Solyom, Balázs, 2016, 2018).

7.1. Non-metallic reinforcements of bridges as internal reinforcements

Within the Project supported by the Hungarian Ministry of Innovation 2018-1.3.1-VKE including high strength, non-corrosive fibre reinforced polymer reinforcement in bridges we would like to present here our preliminary results on application of FRP reinforcement as internal prestressed with CFRP wires (Fig. 19).

Cross-section



Material and experimental properties

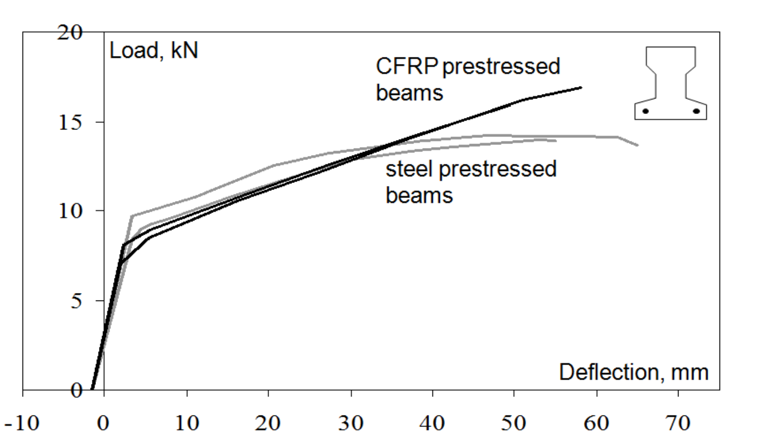


Fig. 19: Comparative study on concrete beams prestressed either with steel or with CFRP wires (preliminary study: Borosnyói, Balázs, 2007)

Our paper (Balázs, Borosnyói, 2000) entitled "*Application of non-metallic (FRP) reinforcements in bridges gives overview*" give further details.

7.2. Non-metallic reinforcements for strengthening of bridges

Concrete structures may need strengthening due to the deterioration of material properties (including excessive cracking or deflection), increase of loads or modification of strengthening system. Fibre reinforced polymer (FRP) can be used in civil engineering for structural rehabilitations and strengthening. The principal reasons for strengthening are deterioration of reinforced concrete elements (electrolytic corrosion) and inadequate serviceability. High tensile strength to weight ratio and corrosion resistance of fibre reinforced polymers provide the attractive for various strengthening applications. FRP can be easily applied and can carry tensile forces of reinforced concrete structural elements. Longitudinal pultruded CFRP strips were used in the herein discussed experiments.

Retrofitting of concrete structures by using externally bonded reinforcements has been published in *fib* Bulletins 14 and 35. By using FRPs for strengthening, two techniques are distinguished (Fig. 20): application of external bonded reinforcement (EBR) (Hollaway, Leeming, 1999; Balázs, Almakt, 2000) or near surface mounted (NSM) reinforcement (Blaschko, 2001; Szabó, Balázs, 2008). Both techniques, especially EBR, gained a wide range of applications in the last two decades.

The externally bonded reinforcement (EBR) consists mainly of gluing of pultruded FRP strips on the prepared concrete surface (Fig. 20c). The near surface mounting (NSM) means application of FRP reinforcement in grooves pre-cut into the concrete cover (Fig. 20a and 20b). The glued surface of strengthening element is doubled (two bonding faces) and relatively free of impurities. Therefore, the strengthening reinforcement in NSM applications becomes more integral part of the element compared to externally bonded applications. The larger bond surface induces better anchorage capacity of the reinforcement and a higher percentage of the tensile strength of FRP can be mobilized.

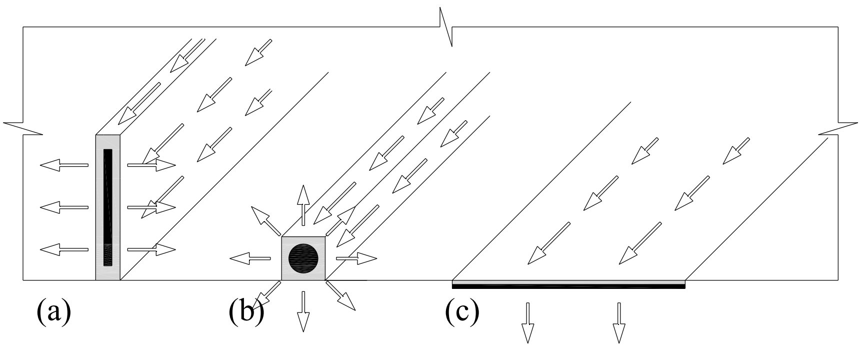


Fig. 20: Interaction forces developed by near surface mounted (NSM) a), b) and externally bonded (EBR) c) reinforcements (Balázs, 2008; Szabó, Balázs, 2011)

An interesting example for bridge strengthening with CFRP EBR in Budapest is presented herein (Balázs, Almakt, 2000). The approach span of Petőfi bridge in Budapest was constructed of precast prestressed pre-tensioned concrete bridge girders with an additional cast in situ reinforced concrete deck (Fig. 21). The first girder suffered seriously from corrosion, i.e. five prestressing strands from twenty-nine were completely corroded in the girder situated under the tram line.

Several ways for strengthening were analysed and finally the decision was taken to bond 5 pieces of 28 m long Sika CarboDur® strips of medium modulus of elasticity. This operation was performed during night after the tram traffic had stopped. The whole bottom flange of the girder was finally covered by a protecting layer against further ingress of de-icing salts into the concrete. Unfortunately, the chloride content of concrete before strengthening was already relatively high and therefore further failure of prestressing tendons is not excluded. Regular deflection and strain measurements are carried out for control.

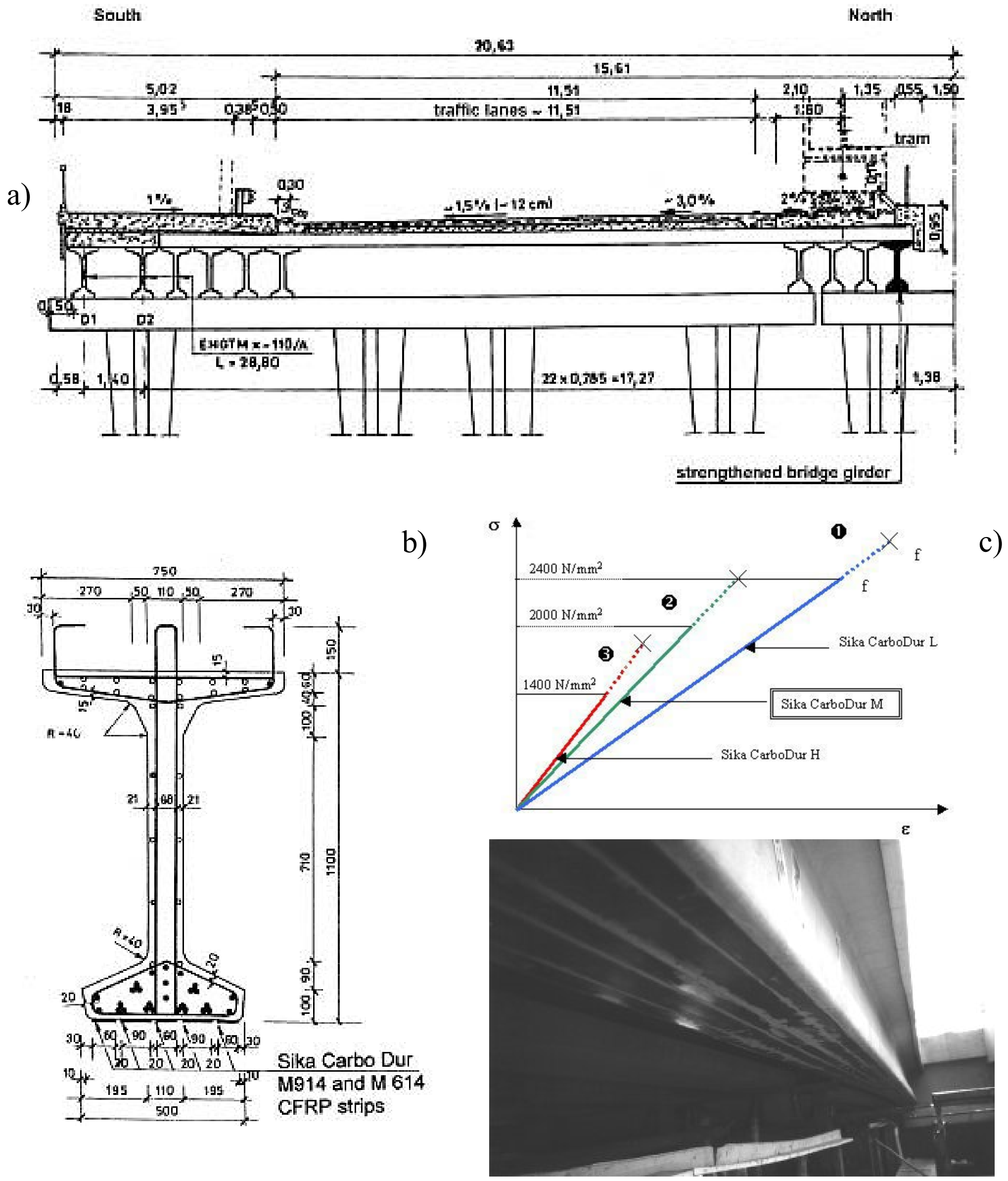


Fig. 21: Strengthening of a prefabricated prestressed pre-tensioned concrete bridge girder in Budapest, Petőfi bridge owing to the corrosion of five prestressing strands (Balázs, Almakt, 2000)
a) Cross-section of bridge; b) Cross-section of strengthened bridge girder; c) Stress-strain curve of strengthening strips

8. CONCLUSIONS

Intention of this paper was to review major developments of bridges as well as to present results of Hungarian Research Grants NVKP_16-1-2016-0019, VEKOP-2.1.1-15 and 2018-1.3.1-VKE.

Developments presented include high strength high performance concrete bridges, lightweight aggregate concrete bridges, high strength, non-corrosive fibre reinforced polymer reinforcement in bridges.

9. ACKNOWLEDGEMENTS

Author gratefully acknowledges the support by the Hungarian Research Grant NVKP_16-1-2016-0019 "*Development of concrete products with improved resistance to chemical corrosion, fire or freeze-thaw*" especially herein for development of freeze-thaw resistance.

Author gratefully acknowledges the support by the Hungarian Research Grant VEKOP-2.1.1-15 "*Development of high performance bridge girder families of normal weight or lightweight aggregates with inclined tendons*" especially herein for the development of lightweight aggregate concrete bridge girders presented.

Author gratefully acknowledges the support by the Hungarian Research Grant Nr. 2018-1.3.1-VKE Project "*Concrete elements with advanced properties*" especially herein for the applications of high strength, non-corrosive fibre reinforced polymer reinforcement in bridges.

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MOBILITY AS A SERVICE: RESEARCH, DEVELOPMENT AND CASE STUDIES IN TAIWAN

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SUMMARY

This paper presents development of Mobility as a Service (MaaS) in Taiwan. First, a strategic planning on MaaS was conducted for development of national policy on MaaS while a dynamic travel information system has been developed with open API for the basic platform of MaaS needs. Implementation approach with the public private partnership was also proposed and assessed in this strategic planning. Based on the results of strategic planning, two MaaS projects are initiated in Taipei and Kaohsiung Metropolitans with different integrated and multimodal transportation services. This paper presents the demonstration projects in Taipei and Kaohsiung, which is part of the National ITS Program. Additionally, a pre-MaaS project implemented in Taipei is evaluated in terms of transportation policy, integration of green mobility, and system performance achieved. This paper concludes by summarizing lessons learnt from the two cases and further research and planning needs for MaaS programs.

Keywords: Mobility as a Service, Sustainable mobility, Urban transportation policy

1. INTRODUCTION

The concept of Mobility as a service (MaaS) is originated from the first trial of subscription-based mobility in Gothenburg Sweden in 2013 and further developed in Helsinki Finland, the modal shift that MaaS promotes, has triggered pilots and trials in major cities globally in a variety of different regulatory and policy environments (Heikkilä, 2014; Pickford, 2018). The implied meaning of MaaS is to build a seamingless and door-to-door transportation service platform that integrates all the information of multiple public transport modes and various feeder services through information technology, and people can use the electronic devices like personal computers, smartphones, or KIOSK to make reservation of this service for organizing their travel plans, based on the schedule of different public transport services. On-demand and customer-made service may also be provided through this platform.

The concept of MaaS has become a crucial policy for sustainable urban mobility. The main objective of MaaS is to build a seamless and door-to-door transportation service platform that integrates all the information of various public transport modes, shared mobility and the first and last mile feeder services through information technology. Therefore, travellers can then use the electronic devices and Apps to plan, organize and pay for their travels and other added activities on their journeys. In this case, travellers are expected to be willing to have a better choice towards the goal of sustainable mobility and liveable city. Jittrapirom et al. (2017) presents MaaS cases being on trial or in operation around the world while Pickford (2018) summarized the state of practice on MaaS. It is shown that about 70 cities around the world are conducting various levels of MaaS implementations and trials while 2/3 of cities are in Europe. Since the MaaS concept originated in Europe, the number of cases is larger than other regions.

While the MaaS development is relatively late in Asian countries, Taiwan utilizes its strong information and communications technology and complete public transportation system in promoting MaaS. It has been initiated two MaaS test cases, the UMAJI in Taipei-Yilan corridor and MenGo in Kaohsiung. Ministry of Economy, Trade and Industry and Ministry of Land, Infrastructure, Transport and Tourism in Japan have also announced their joint project "Smart Mobility Challenge" in which regions and cities are encouraged to cooperate in implementation of new mobility services taking advantages of IoT and AI applications in societies (METI, 2019). Both Ministries will initiate pilot projects and demonstration tests of MaaS in various cities and regions. It is expected to develop role models and business plans for the collaborations on new mobility services. It is also observed that the Ministry of Land, Transport and Tourism of Japan has published the list of the selected 19 of the presented projects across 28 regions to be publicly supported, tested and evaluated from in the coming few months. In a very well design and planning, none of the supported projects will be carried out in Tokyo. All of the 6 services in urban areas will be developed in secondary cities while 5 will focus on providing services to rural areas. And, 8 services will be exclusively focused on tourism related applications where car usage is on the rise (MLIT 2019).

This paper mainly presents the MaaS development in Taiwan. Efforts on national strategic planning and dynamic travel information platform are presented. Specific demonstration projects implemented in Kaohsiung and Taipei are analyzed and compared. Challenges and lessons learnt from the two demonstration projects are discussed and summarized. Further research and planning needs are also proposed.

2. NATIONAL POLICY ON MaaS DEVELOPMENT

In development of MaaS projects, a strategic planning on MaaS was conducted by Institute of Transportation in Ministry of Transportation and Communications for formulation of national policy, identification of benefits and challenges, and recommendation of implementation strategy (Chen et al., 2017). And, a “Travel Information Platform” has also been proposed and used for the development of on-demand service in a national level with open data policy and smart community program (National Development Council, 2018).

This strategic planning on MaaS was funded by National ITS Program (2017~2020) in which six sub-programs were proposed: (1) Traffic Safety Program, (2) Integrated Corridor Management Program, (3) Rural Area ITS Applications Program, (4) Mobility as a Service Program, (5) Connected and Automated Vehicles Program, and (6) ITS R&D Program. In a policy level, the strategic planning has identified the importance and benefits of the National MaaS Plan that consisting of five economic features, namely, intelligent economics, technology economics, transport economics, sharing economics and service economics. An operational scheme with public private partnership framework is also proposed for the MaaS demonstration projects in this strategic planning.

Based on the results and suggestions of the strategic planning, two demonstration projects were initiated in Spring of 2018 while the following KPIs have been used for performance evaluation: (1) Behaviour Change, (2) Travel Satisfaction, (3) Service Quality, (4) Integration of Service Providers, (5) Number of Horizontal Alliances, (6) Economic Benefit, and (7) Financial Benefit. MOTC has also applied the project management system to guarantee the two MaaS demonstration projects to be on schedule, in line with the quality and budget as planned and having certification from IV&V organization (Lin, et al., 2019).

In additional to this national strategy, Travel Information Platform is considered crucial for success of MaaS and was originally proposed to develop in a national smart community program. However, the scope and contents of TIP has been comprehensively discussed due to the privacy and security concerns. Therefore, Ministry of Transportation and Communications and National Development Council decided to start with system on public transportation information rather than the general travel information. Therefore, a multi-year project “Public Transportation Data eXchange Platform, PTX” is then formed and has become a real time open data hub for all kinds of public transportation services in Taiwan since 2017 (Hsiao, 2018).

The main strategic objectives of PTX have been designed as follows: (1) Enhancement of passenger service quality, (2) Improvement of operation productivity (efficiency increase and cost reduction), (3) Enhancement of decision making quality, and (4) Increase of economy (e.g., App development and added value of PTX as well as innovative service based on PTX). The original PTX is consisted of high speed rail, conventional rail, urban rail, trams, freeway bus, regional bus, city bus, public bike, ferry and aviation. In recent two years, it has included general traffic information for both static and dynamic information by integrating data from e-tag of Electronic

Toll Collection system, CCTV, vehicle detection, and parking information, as shown in Fig. 1. Fig. 1 also shows the contents of all information and the time line of including additional travel information. Additionally, information for tourism and weather conditions is gradually included in this platform for preparing to have more added value and applications of this information platform. Latest achievements of PTX can be summarized as follows:

(1) More than 1,000 registered users, half of them are value-added organizations and/or companies,
(2) More than 550 API services provided,
(3) More than 80% of data source organizations cooperate with PTX, and
(4) Average 2.5 million calls per day, 500 million data sets requests, and 350 GB download.

Therefore, with the collaborative effort done by MOTC, IISI Group and Advanced Public Transportation Research Center in NTU, PTX has excellent data quality and reliability (Hsiao, 2018). PTX and its extended version has become an excellent tool for on-mobility services and other applications.

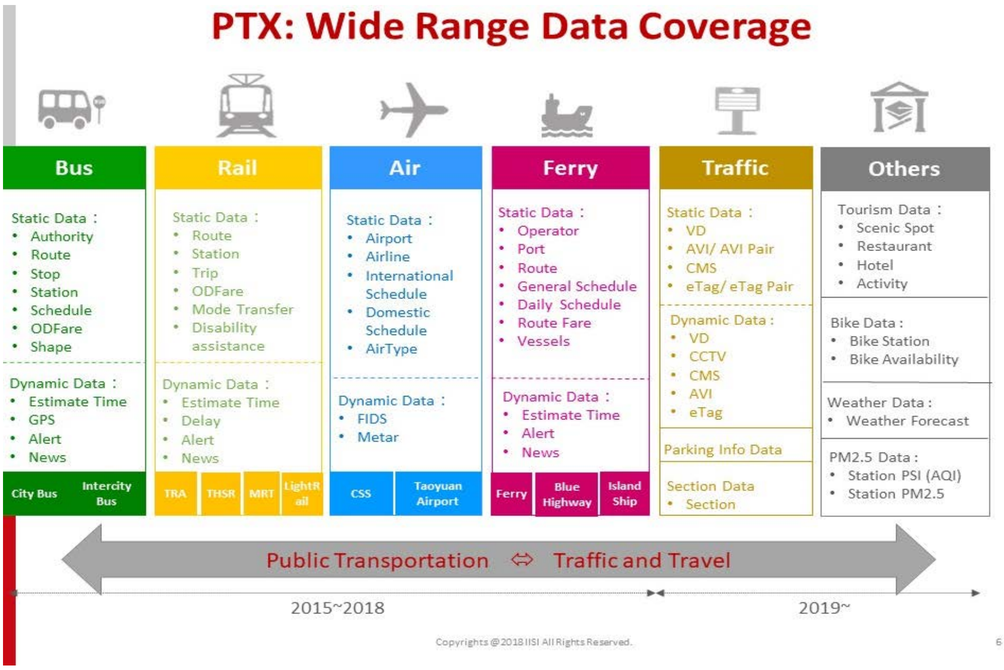


Fig. 1: Various Data Sources in PTX

3. MaaS DEMONSTRATION PROJECT IN TAIPEI

As mentioned, two national demonstration projects are formulated with the financial support of National ITS Program. The first one is implemented for 40 km Taipei-Yilan corridor in which MaaS project is designed for alleviating traffic congestion in the corridor by providing integrated public transportation systems as well as a mobile platform (UMAJI) for trip planning and payment services. It is expected that the existing 63% share of private motorized vehicles may have a significant modal shift to public transportation with good integration of public transportation systems and innovative and shared feeder services in the trip ends in Taipei and Yilan (Chang, 2018; Liu, 2018). Fig. 2 shows the main features of the App UMAJI proposed in this MaaS projects.

It is also worth noting that a consortium is formed by public private sectors. The consortium is selected through a bidding process to obtain the concession from Ministry of Transportation and Communications for planning, design and implementation of the MaaS demonstration project with public fund in the first stage. It is expected to come out a financial plan and business model for sustainable operation by the consortium in the 2nd stage of the project in 2020 and then have the financially sustainable operation from mid 2021. Tab. 1 shows the partners of this MaaS Project in Taipei. It can be observed that one of the biggest telecom companies ChuHwa Telecom is the team leader while system integrator, data analytics expert group, operators of freeway bus, railways and public bike systems, and payment service provider are invited as partners in this project.

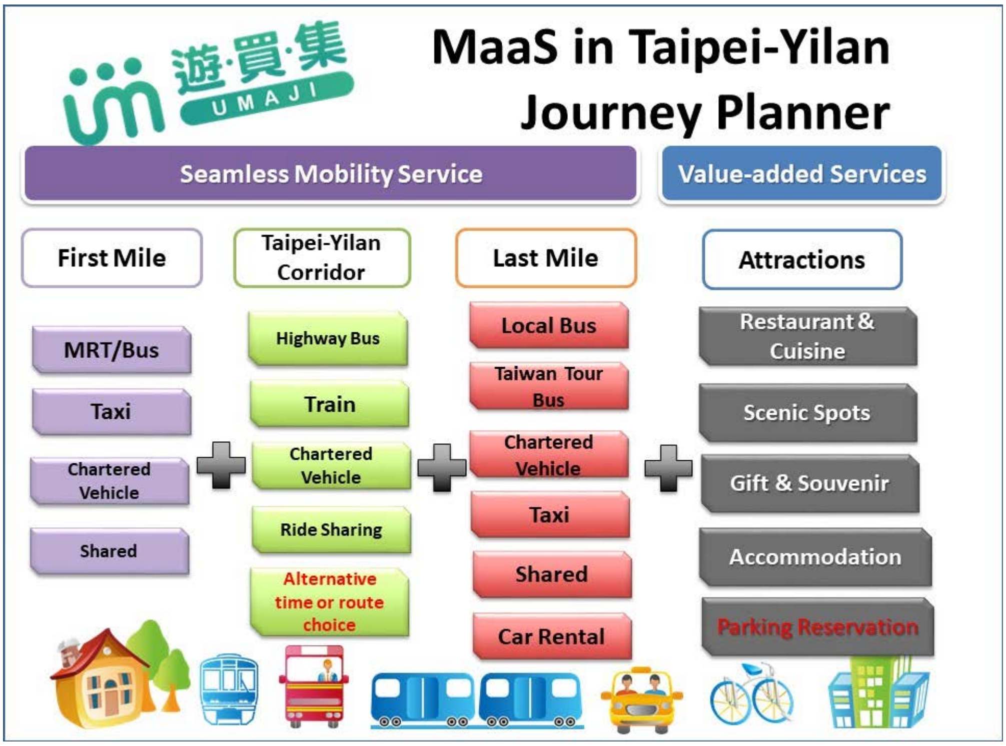


Fig. 2: Key Elements of Platform "UMAJI" in Taipei-Yilan MaaS Project

Tab. 1: Partners in UMAJI (Taipei-Yilan MaaS Project)

Company Name	**Functions**	**Features**
ChungHua Telecom	Project Leader	Public Own Corp
IISI Group	ICT and System Integration	III + ChungHua + IBM
KingwayTek Technology	Spatial Information, Data Analytics and Digital Map	Private Sector (with ChungHua Telecom invested)
Public Transport Operators	Taiwan Railway Adm, Freeway Bus, City Bus, Taxi, Car Rentals, Public Bike	Public Sector: Taiwan Railway Private Sector: Bus Operators, Taxi Co., Car Rentals, Public Bike
EasyCard	Multi-Media Payment Platform	PPP Company (Private 60%, Public Sector 40%)
Research Institutes	Universities, Research Institutes	Public Sector

In addition to Taipei-Yilan Corridor MaaS project sponsored by the central government, Taipei City Government has conducted a feasibility and preliminary trial on Monthly Payment Scheme since 2017. From April 2018, the Monthly Payment Scheme, also being considered as a metropolitan wide Pre-MaaS program, has started for promotion in which Bus, Urban Rail (MRT) and Public Bike (U-Bike) are included. Travellers pay USD43 for a monthly pass can use all of these three services, Metro, Bus and Public Bike. It is estimated that this scheme needs a total amount USD32 millions for subsidy, which will be innovatively provided by the City Parking Management Fund.

The pre-MaaS program has been evaluated for its operation in the first three month. It is shown that 2% of total public transportation ridership has increased for the first three month trial of the monthly pass compared with market share of daily trips shown in Tab. 2. It has also shown in the 2nd season that it has been achieved to 3.2% increases in public transportation systems while more than 80% of the increased trips are from the existing users (Chen, 2019). This performance is less than expected and Taipei City Government has been asked by City Council to review it and propose promotion campaign for increasing its performance and impacts.

Tab. 2: Transportation Systems and Market Shares in Taipei Metropolitan

Mode/ System	System Descriptions	Daily Trips (Market Share)	
		Metropolitan[1]	Core City[2]
MRT	142 km with 120 stations	2.21 Mi (16.9%)	1.01 Mi (18.7%)
Bus	Operators: 16, Fleet: 6,400, Routes: 330	1.76 Mi (13.5%)	1.10 Mi (20.4%)
Taxi	Fleet: 53,500	1.12 Mi (8.6%)	0.62 Mi (11.8%)
Public Bike	Fleet: 36,000 Stations: 840	0.22 Mi (1.7%)	0.14 Mi (2.5%)
Car	2.5 Mi (Core City[2] 0.95 Mi)	3.23 Mi (24.8%)	0.91 Mi (16.8%)
Motorcycle	3.2 Mi (Core City[2] 1.05 Mi)	4.95 Mi (38.1%)	1.22 Mi (22.6%)
Others (Walk, Bike…)	--	0.66Mi (5.1%)	0.39Mi (7.2%)

Note: 1. Taipei Metropolitan: Population 6.8 millions; Region Area: 3,100 km^2; 2. Taipei City (Core Area): Population 2.8 millions, City Area: 272 km^2.

With this comparatively small effect on public transport ridership, this pre-MaS has also been discussed and argued on various issues such as pricing scheme, subsidy and integration with the MaaS solutions. In terms of pricing scheme, Chang (2018) proposed that the payment of monthly pass needs to be reduced from USD43 to USD30 if the policy of monthly pass is to attract more users shifted from private motorized vehicles, particularly motorcycles. It's been found that the average user cost for motorcyclists in Taipei Metropolitan is 1 USD (Chang, Chang and Chen, 2016). It is almost impossible to attract motorcyclists to use the monthly pass of public bike and public transportation services. It is suggested that the pricing scheme for monthly pass and future MaaS system may be further explored and designed based on analysis of willingness to pay (Ho, Hensher, Mulley and Wong, 2018; Chen, Chang, Chen, 2018). It is also suggested that the Taipei-Yilan Corridor MaaS program should be integrated with the monthly pass scheme and become metropolitan-based MaaS program in the next stage.

It is also worth noting that the monthly pass for the three green modes is only part of comprehensive transportation policy announced and implemented in Taipei City Government (Taipei Core Area). The current 60.6% share of green mobility in the city core area is expected to sustainably increase and achieve the goal of 70% by 2025 with the policy of Safe, Green, Smart and Shared Mobility (Chen, 2019).

4. MaaS DEMINSTRATION PROJECT IN KAOHSIUNG

Kaohsiung is the third largest municipalities in Taiwan, with population around 2.77 million and area of 2,952 square kilometres. There is a variety of public transportation systems in Kaohsiung, including city buses, urban railways (Mass Rapid Transit and LRT), ferries, and public bike (C-Bike system). According to the 2016 National Travel Survey (MOTC, 2016), Kaohsiung had the second lowest market share of public transportation among six municipalities, which was 9.3%. Although Kaohsiung is the only city that has MRT system other than Taipei Metropolitan Area, the market share of MRT was 2.2%, which was much lower than that of Taipei Metropolitan. Moreover, the market share of scooters was the highest one among all modes with 60.8%.

Therefore, to boost the use of public transportation in Kaohsiung is one of the objectives to activate the demonstration project of MaaS, which is also the first pilot project of MaaS in the scope of metropolitan. Based on the checklist provided by Li and Voege (2017) for assessing whether the MaaS service can be implemented in a city, Kaohsiung indeed has enough and diverse transportation modes, and most of the transportation operators has been providing their real-time service information. A preliminary study on willingness to pay for MaaS in Kaohsiung has also shown it is feasible for various types of passengers to join MaaS system (Chen, Chang and Chen, 2018). Besides, passengers in Kaohsiung are also able to access all transportation services through e-ticket iPass and in some cases Easycard, as shown in Fig. 3. It is worth noting that both of these e-ticketing smart cards are e-purse and will become a mobile payment system that will enhance the capability for MaaS applications.

Fig. 3: E-ticketing Systems of iPass (left) and Easycard (right)

According to Chen (2018), three service concepts were adopted in the development of MaaS in Kaohsiung, namely, friendliness, integration, and trusts, so-called FIT. The FIT concept corresponds to theoretical findings and suggestions in some of previous studies on MaaS.

Kamargianni and Matyas (2017) defines MaaS as a "user-centric intelligent mobility service distribution model in which all mobility service providers' offerings are aggregated by a sole mobility provider, the MaaS provider, and supplied to users through a single digital platform", which includes the friendliness and integration concept. As for the trust concept, Li and Voege (2017) also stated that MaaS service should ensure the quality of information provision and service offering in order to maintain the confidence of users toward the on demand mobility service.

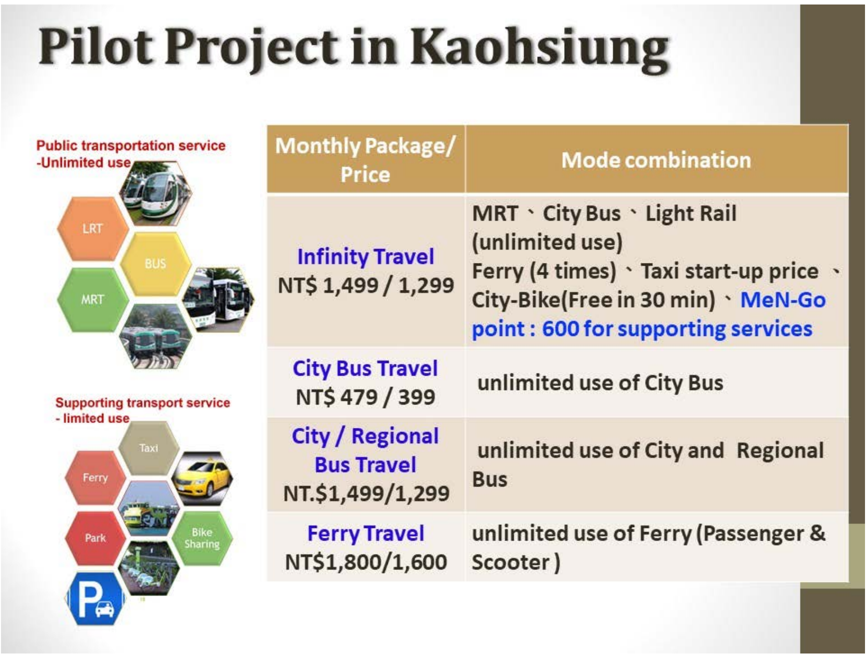
Furthermore, two population segments were targeted by the MaaS service. One was student from 17 to 22 years old and the other was white-collar commuter in specific trip attractions, such as harbour area and scientific parks. The reason why students are targeted is that they do not have private motorized vehicles; even they have, they are in high risk of engaging in fatal accidents. Therefore, providing seamless public transportation and feeder services is also to aim a safe transportation environment for young generations and our societies.

Compared to Taipei UMAJI, the consortium has created "MenGo" as the brand name and App of MaaS in Kaohsiung (Chen, 2018; MenGo, 2019). As mentioned, the Kaosung MaaS is a collaboration of public sectors and private sectors. The Institute of Transportation of MOTC and the Transportation Bureau of Kaohsiung City Government are from public sector while the consortium consists of InfoChamp Systems Corporation, iPass Corporation, and various transportation service providers. The consortium obtains the concession for planning, design and operation in early 2018 while it has started its official operation in Kaohsiung on September 2018. App MenGo can be downloaded and used on both iOS and Andorid systems. Users can use this App to subscribe to a service package, look for traffic information and plan their trips. Fig. 4 shows the basic functions and framework of Mengo. In the near future, more of value-added applications will be included in the 2nd stage in 2020.

Fig. 4: Functions and Framework of Mengo in Kaohsiung MaaS

As planned, the transportation services currently integrated by MenGo in Kaohsiung include mass transit (city buses, inter-city buses, MRT, light rail, ferries) and shared transportation (public bicycles). However, when users arrange their trips through the smartphone application, besides the above transportation services, there will be arrangements for the use of Taiwan Railways and the High Speed Rail. Users can also choose to connect to the applications of taxi operators for taxi pooling and/or other car pooling services.

At present, MenGo offers four kinds of monthly service packages for users to choose from, as shown in Fig. 5, which are the "Infinity Travel", "City Bus Travel", "Ferry Travel" and "City Bus + Inter-City Bus Travel", respectively. Students can naturally subscribe at a discounted price. Furthermore, a loyalty scheme has been designed for MenGo users. Those who subscribe to the Infinity Travel Package are rewarded as additional 600 MenGo points. These MenGo points have period of validity of one month and can be used to discount taxi fares in which one MenGo point can discount one dollar of taxi fares. In addition to discounting taxi fares, it is planned that MenGo points will also discount shared scooter fares and parking fees in the 2nd stage of MaaS project.



Monthly Package/ Price	Mode combination
Infinity Travel NT$ 1,499 / 1,299	MRT、City Bus、Light Rail (unlimited use) Ferry (4 times)、Taxi start-up price、City-Bike(Free in 30 min)、MeN-Go point : 600 for supporting services
City Bus Travel NT$ 479 / 399	unlimited use of City Bus
City / Regional Bus Travel NT.$1,499/1,299	unlimited use of City and Regional Bus
Ferry Travel NT$1,800/1,600	unlimited use of Ferry (Passenger & Scooter)

Noted: 1US$ = 30NT$

Fig. 5: Service Packages of MenGo in Kaohsiung MaaS, Source: MenGo (2019)

Users of MenGo can subscribe the service packages through both of the smartphone application and official website. However, unlike most of the MaaS cases, users of MenGo cannot access the transportation services directly through the smartphone application after subscribing to the package in this stage. An e-ticket such as iPass Card or MenGo Card will be activated and tapped to access the transportation services that MenGo has integrated for applications of an e-ticket system. It might be less convenient than using smartphone applications to access multimodal transportation services. It is believed that MenGo has cooperated with Dream Mall, which is the largest department store in Kaohsiung, to provide significant discounts when people pays with MenGo Card in Dream Mall. The MenGo Card is shown in Fig. 6.

Fig. 6: The MenGo Adult (left) and Student (right) Cards, Source: MenGo (2019)

5. PRELIMINARY EVALUATION OF MaaS PROJECT IN KAOHSIUNG

According to Chen (2018) and a latest performance evaluation, it has been shown that MenGo has sold 15,492 service packages from Sept 2018 to Dec 2018 while 94.4% of users keep on using MenGo in this three-month period. It keeps increasing in terms of service packages and three-month pass. It is worth mentioning that the users of motorized vehicles contribute 21% of MaaS MenGo members. It is implying that a total 58,800 trips per month shifted from cars and/or motorcycles to green transportation. The situation complies with the findings from Mulley, Nelson, and Wright (2018) that service packages can lock in customers and can help gain profit of sustainable mobility. Therefore, it is suggested that the contents of service packages should be reviewed on a regular basis to understand the preferences and choices of users and enhance the overall packages of MaaS. Based on the preliminary performance, Kaohsiung MaaS has demonstrated there is great potential of attracting more private motorized vehicle users to take public transportation and shared mobility for achieving the ultimate vision of sustainable mobility.

Besides the content of service packages, the pricing scheme should also be re-evaluated as well. Currently, the pricing scheme is set according to the two targeted groups, which are students and white-collar commuters. However, Ho, Hensher, Mulley, and Wong (2018) found that the level of acceptance of MaaS varied significantly across population segments and suggested that MaaS operators may need to carefully segment the market and adopt a cross-subsidy strategy to achieve a commercially operational level. Therefore, it is suggested that MenGo can keep exploring possible beneficial population segments to maintain its financial sustainability, especially as it is stepping into the phase of self-funding and financially sustainable operation in late 2020. Additionally, it is also feasible to explore more population segments than the current two segments in order to have more comprehensive pricing scheme for the purpose of financial sustainability.

In terms of functions, MenGo has integrated public transportation such as buses, MRT, LRT, and ferries, and C-Bike. However, in other MaaS cases like Whim and UbiGo, taxis, shared cars, and rental cars are also included in their service offerings. Hensher (2017) indicates that point-to-point and point-via-point-to-point services are important factors in MaaS framework while Sochor, Strömberg, and Karlsson (2015) also pointed out that shared mobility resources are crucial to reduce the use of private motorized vehicles. It has been observed that 21% of MenGo users were originally private motorized vehicle users. In order to encourage more modal shifts, it is advised that MenGo should be cooperated with taxi and shared car operators in providing the do to do service. In addition, due to local characteristics of the high market share of scooters in Kaohsiung, a shared scooter system should also be considered in MaaS design for next stage. It has also been mentioned by Kaohsiung City Government that IOT, AI and shared mobility may have great potentials in the coming few years. It is also confirmed that electric scooters and electric cars may also be included for green mobility in the 2nd stage in 2020 (Chen, 2018).

Kamargianni and Matyas (2017) proposed a framework of the MaaS business ecosystem, detailing the roles of each of the participants. The business ecosystem refers to a network that affects how the consortium creates and captures value and the business ecosystem for future MaaS. It has been indicates that MaaS service operators could be either a public transportation sector or private company. However, since MenGo is a collaboration of the central and local governments as well as a consortium consisted of various partners in mobility services, expected goals for implementation of MenGo in the first stage can be achieved in a relatively short period of time.

The big challenges for implementation of MenGo are legally and geographically restricted and is not able to provide "transportation roaming" service, which refers that travellers can enjoy the same transportation service any where the MaaS service is provided by the integrated service operator. With these considerations, Li and Voege (2017) suggests that MaaS service providers in which only local service might have limited room for profitability. Therefore, it is again indicated the importance for MenGo to seek other way to generate revenue for reaching financial sustainability.

Kamargianni and Matyas (2017) also stated that the role of ticketing and payment solution providers of MaaS a business model of ecosystem. iPass Corporation has been playing such role in the business model of MenGo. Although it is mentioned above that currently the transportation service integrated by MenGo must be accessed through e-ticket like iPass Card or MenGo Card. iPass Corporation shall be providing the service that people can enter the Kaohsiung MRT system with mobile tools such as QR code, as shown in Fig. 7. Thus, in the foreseeable future, it can be expected that iPass Corporation may also provide QR-code payment and ticketing services for MenGo, and users can access all kinds of mobility services with their smartphone applications. Fig. 7 also shows that passengers could have 10 loyalty credits by using this QR-code payment scheme for Metro.

Fig. 7: iPass Corporation's QR-code Ticketing Service for Kaohsiung MRT, Source: LINE Pay iPASS (2019)

6. CONCLUSION AND RECOMMENDATION

This paper presents development of Mobility as a Service (MaaS) in Taiwan from the perspectives of national and city levels. This paper also identifies the importance of a strategic planning for the MaaS policy and implementation strategies. MaaS demonstration projects in two cities haven been presented and compared in this paper. It is shown that planning and design of MaaS system need to consider local transportation networks, operations and services. Both MaaS demonstration projects are initiated by public fund in the National ITS Program with a Public Private Partnership approach. A role business model for financially sustainable operation is expected to formulate in the 2^{nd} stage of these two projects.

It is also observed that MaaS project has an external benefit which is not easy to be internalized. Therefore, public sector has obligations to provide enough resources for a necessary service for considering the external effects and the potential subsidy schemes.

This study also evaluates MenGo in terms of its demand side, supply side, and governance and business models, according to other MaaS cases around the world, such as Whim and UbiGo. Since the MaaS system and local characteristics are closely related to each other, it is suggested that future research and planning could focus on understanding the willingness of various users to pay for the variety of MaaS functions in order to have an optimal design of MaaS.

Lastly, Taipei MaaS Project has also shown that there is a big challenge for telecommunication industry and IT groups to play as an integrator of mobility service provider. A fusion scheme for a better collaboration among IT and transportation partners should also be proposed and assessed. Furthermore, the existing MaaS systems can be re-evaluated to propose improvements and measures to continuously attract users from private motorized vehicles to MaaS services.

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ELASTIC ELEMENTS IN TRACK INFLUENCING TOTAL TRACK COSTS AND REDUCING VIBRATIONS

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SUMMARY

Under Sleeper Pads (USP) are associated with having a positive impact on track behaviour of ballasted track and reducing vibrations. Elastic products are also suitable for slab track systems, where they add elasticity and enable higher quality of living for nearby residents.

1. IMPACT OF UNDER SLEEPER PADS ON TRACK SUSTAINABILITY

Starting with the impact on track behaviour one question should be addressed: Why do concrete sleepers need further development? The implementation of concrete sleepers gave track a much better stability in increasing the lateral track resistance and thus lowering the maintenance demand. The one world record run in 1955 with 331 km/h destroyed the wooden sleeper track. There were two main reasons: the little lateral track resistance due to the light superstructure and the manual maintenance not allowing continuity of track quality, neither in track laying nor within track maintenance.

These problems were solved impressively long before Under Sleeper Pads were in use by implementing mechanised track renewal and maintenance and heavy superstructure with 60 kg rails and concrete sleepers. After the world record run in 2007 with 574.8 km/h track was opened for operation without any maintenance action necessary. Furthermore, concrete sleepers are cheaper than wooden ones and show a longer service life. Thus not only loads and speeds could be increased but also life cycle cost of superstructure was reduced by using concrete sleepers.

However, concrete sleepers show still one main disadvantage compared to wooden ones: the contact area between sleeper and ballast is very small. Tests using a black paper below the sleepers allowed identifying the contact area between ballast and sleeper after tamping. The used ballast in Austria has a maximum size of 63 mm. The use of a dynamic track stabiliser (DGS) showed significant differences in the results as presented in Fig. 1.

CONTACT AREA	wooden sleeper	concrete sleeper	concrete sleeper with USP
without DGS	9% to 10%	2% to 8%	up to 15%
using DGS	up to 18%	up to 15%	up to 35%

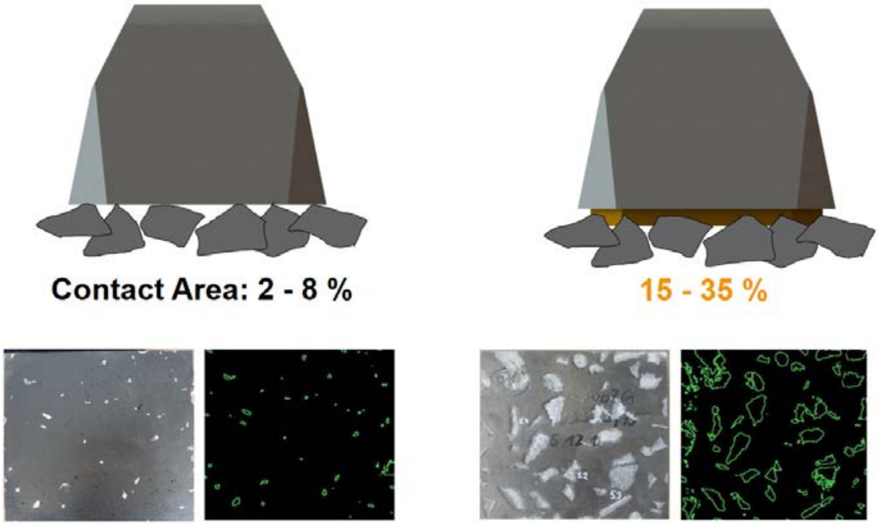

Fig. 1: Contact Area Sleeper – Ballast; left: concrete sleeper, right: padded concrete sleeper

On the one hand the result show the effectiveness of track stabilising, on the other hand the high ballast degradation using concrete sleepers is evident. The hard-to-hard situation in this contact area causes only a few, highly overloaded contact points forming the starting points for a limited number of "force-paths" through the ballast structure rather than allowing the entire ballast structure to act as a sleeper-support. Edges and corners are quickly cracked off, leading to these unpredictable initial settlements. As the number of "force-paths" varies under neighbouring sleepers, the sleepers settle differently, causing initial errors – equal to a reduced initial quality – and increasing the rate of deterioration. To sum it up: The bigger the contact area, the smaller the initial settlement and thus the differences between these settlements - the higher the contact area between sleeper and ballast the higher the initial track quality. These results have been very promising from the technical point of view. However, installation of Under Sleeper Pads cause additional investment costs. To analyse the efficiency of this additional investment from the economic point of view the technical effects (track quality behaviour) must be analysed over the entire service life of track (lice cycle costing) as just a life cycle approach allows evaluating the economic efficiency of various track types. In order to do so behaviour of track must be analysed, as understanding of track behaviour is the precondition of forecasting it and thus evaluating life cycle behaviour and life cycle cost of track. Within an extensive research program at Graz University of Technology in cooperation with the Austrian Federal Railways Infrastructure, track behaviour over time was analysed.

All results described in the following are based on extensive data of the Austrian Federal Railways. Testing Under Sleeper Pads already started in the late 1990s, the implementation of USP at ballasted track in Austria began roughly 20 years ago. The tests and the analyses of the change of track behaviour lead to general implementation of under sleeper pads for concrete sleepers at Austrian Federal Railways. The respective regulations are in power since 2007. Fig. 2 depicts the implementation of Under Sleeper Pads in the network of the Austrian Federal Railways over time. There are still some wooden sleeper installed e.g. in sharp curves, sidings, and branch lines. Concrete sleepers without USP are built in mainly in case of single sleeper exchange and short sections of track renewal. The reason is to avoid too many places where superstructure with and without USP meet, as these two types of superstructure show significantly different behaviour.

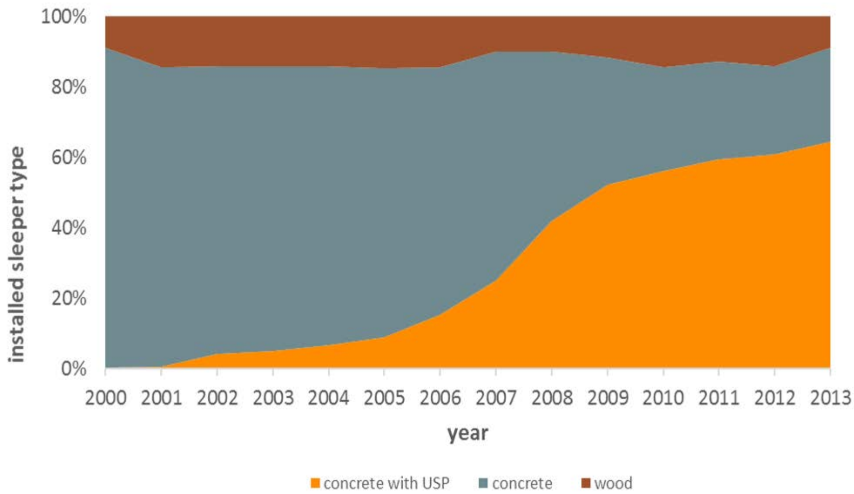

Fig. 2: Implementation of Under Sleeper Pads at Austrian Federal Railways

Track behaviour is evaluated in analysing time sequences of track recording data. In case of USP, up to 2017 already 60,000 sections with USP are compared with respective sections without USP. It can be shown that track deterioration is cut in half with regards to levelling-lining-tamping. This is caused by an increase of the contact area between the sleeper and the ballast bed. Measuring in track and lab shows up to three times higher contact areas using USP. This reduces the stresses in the ballast bed. Furthermore, it also has a very positive impact on track service life as in case of good subsoil worn out ballast is limiting track service life. In Austria, there are also some hundreds switches equipped with USP.
Based on the analyses of track behaviour a life cycle cost evaluation was conducted taking the above described effects on track behaviour into account.

2. TRACK QUALITY BEHAVIOUR

Describing track quality behaviour various boundary conditions as transport volume, type of superstructure, quality and status of all components, alignment (radii), ballast quality, quality of sub-layer as well as sub-soil, the functioning of the dewatering system, position of stations, bridges and turnouts must be taken into account. Therefore, a data-warehouse was set up covering track recording car data (initial status, present status, and quality figures), type and age of superstructure and sub-structure, and transport load. The research was based on these data for the main railway net of Austrian Federal Railways covering time sequences of already 17 years (Fig. 3).

This structure allows comparing different types of superstructure for a big number of sections facing the same boundary conditions. The comparisons can be done every 5 m for a data set of 4,000 km in total and thus provides a big number of comparable sections. This allows identifying the effects of initial quality as well as calculating the specific deterioration rate for a given set of boundary conditions and different types of superstructure.

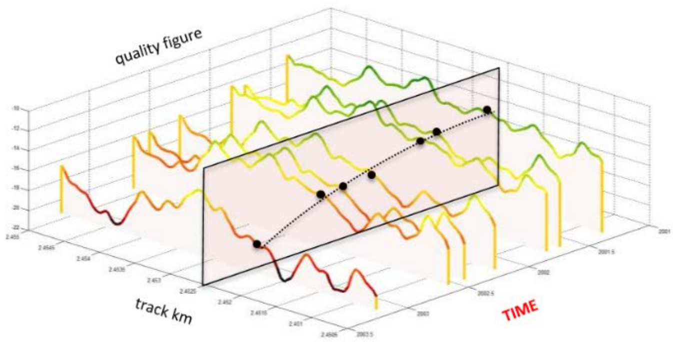

Fig. 3: Analyses of Track Behaviour over Time

The results are very promising as the ballast is identified to be the element limiting the economic service life of track. This underlines the expectations of Under Sleeper Pads as stresses in the ballast bed are reduced due to the increasing contact area between sleeper and ballast (refer to Fig. 4).

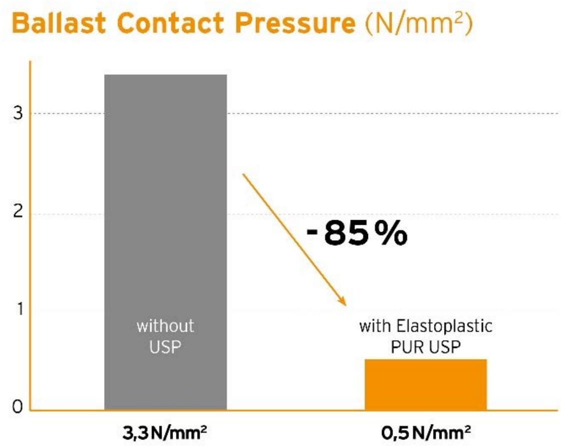

Fig. 4: Reduction of ballast contact pressure with USP

3. LIFE CYCLE COSTING

Life Cycle Costing (LCC) is a process whereby costs of various project alternatives are considered for building and maintaining an asset. These alternatives must all provide the same level of service and benefits to be effectively compared using LCC. Total costs under the LCC methodology are initial design respectively the costs of re-investment and construction, ongoing maintenance costs, and scrap value or disposal cost for the previous asset. All these costs are gathered for each alternative. Then the user discounts the costs back to present day Euros and summarises them. Engineering departments typically have problems with convincing management to go with a higher initial cost in order to save on maintenance or to allow longer service lives. The LCC analysis plainly diagrams the total cost which may aid in management discussions.

However, there are other decision factors that need to be addressed: risk, future availability, best maintenance practices, and environmental concerns, just to mention a few that may be unique to a certain project. Some of these factors may possibly be addressed by increasing future costs. As LCC addresses only those decision factors that can be stated monetarily costs of operational hindrances become important in describing costs of reduced availability.

Application of LCC techniques provides management with an improved awareness of the factors that drive cost. Thorough analysis of the construction and the ongoing maintenance are outlined in detail so as to make omissions more obvious to the LCC creators. It is important that the cost drivers are identified as completely as possible so that the ultimate decision makers can make the most informed decision.

Two attributes of permanent way make life cycle costing especially useful in the field of railway infrastructure: an extremely long service life and a strong relation between initial quality, maintenance demands, and service life. Investment determines the initial track quality, while maintenance affects future track quality and service life. Both investment and maintenance strategies must be considered together, as focusing on investment strategies or isolated maintenance regimes will lead to sub-optimal decisions.

It is difficult to predict the total life cycle cost of long-lasting assets because of the likelihood that there will be unexpected changes related to component costs and maintenance productivity. Moreover, LCC analyses usually focus on the factors and cost categories that are most affected by the alternatives that are being investigated. Therefore, the LCC values that result from a study are not necessarily complete or accurate or suitable for decision-making.

It is much better to compare different options by looking at the differences in life cycle costs. These differences can be directly tied to the features that are expected to differ among the various options. The best option will be the one that is expected to have the greatest reduction in LCC from the base case. Note that the LCC is expressed as an annuity (i.e. as an equivalent annual cost over the life of a project). Sensitivity analyses are generally conducted as part of life cycle cost evaluations in order to attach critical values for sensitive input data.

The discounting rates depend on the service life of the project calculated. Discounting rates of maximum 5 per cent net are generally in use regarding service lives of 30 and more years.

One of the most important figures within LCC is the service life. Unfortunately the service life is not a given fixed value, but influenced to extremely by the initial quality and the maintenance executed. Furthermore, it must be differed between the technical and the economic service life, as the technical service life can be increased by expensive maintenance actions as single sleeper exchange or by limiting the operation in speed and/or axle load. However, this must not be the target as it leads to high costs and poor performance. The economic service life is the time span resulting in the annual cost. The economic service life is reached as soon as the additional maintenance necessary to prolong service life is more costly than the reduction of depreciation due to the prolonged service life.

It can be mathematically proven that the ratio of the maintenance intervals is indirectly proportional to the one of the deterioration rates. In other words, this means that a halved deterioration rate leads to a doubled interval for tamping actions and points out the importance of the rate of deterioration. Furthermore the research showed, that increasing maintenance cycles due to high quality leads to a remarkable increase of service life. However, if the maintenance level is reduced leading to poor quality, service life will be shortened. These effects depict the overwhelming importance of track quality, namely the initial quality and the deterioration rate, for service life, maintenance demand and thus life cycle cost of track.

4. RESULTS COMPARING STANDARD TRACK AND TRACK WITH UNDER SLEEPER PADS

As the deterioration rate b differs widely due to the various boundary conditions, comparing track with and without USP requires comparing these two types of ballasted track facing the same boundary conditions. Elastic footings for concrete sleepers are under investigation since quite a long time throughout Europe. It is obvious from the testing results that track deterioration is reduced dramatically due to the use of Under Sleeper Pads (Fig. 3).

The general results showed that:

- the initial quality is increased by 18 per cent,
- the tamping cycle can be prolonged without loss of quality in a range from 2.00 to 3.00, and
- the service life should increase by more than one third.

The initial quality of USP tracks is better than the one of conventional tracks as the initial settlements of track are reduced and with reducing these settlements their absolute differences – the initial errors – are reduced automatically.

These data form the input data for calculating the economic efficiency of USP. The calculation is based on Austrian cost figures. Additional cost of USP is 30 per cent of sleeper costs including fastenings. Not to fix the results on a specific cost level numerous sensitivity analyses had been carried out in varying specific cost data.

The economic evaluation shows that this reduction leads to enormous savings in terms of total LCC justifying the relatively low additional investment cost. However, savings are the higher, the higher traffic loads are (Fig. 5). Therefore the Austrian Federal Railways (ÖBB) started to implement USP equipped concrete sleepers as standard solution on highly loaded tracks (more than 30,000 daily gross tons per track), sharp curves (less than 600 m), high speed (more than 160 km/h), and turnouts, first.

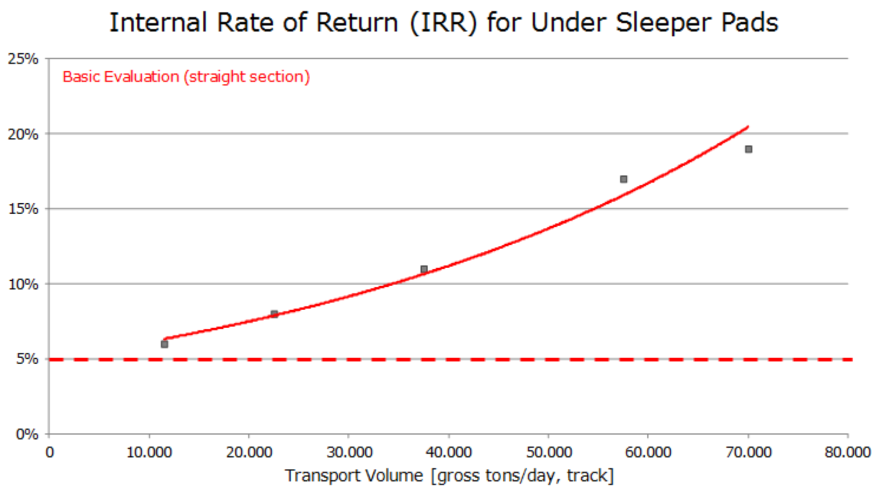

Fig. 5: Internal Rate of Return for the Additional Investment into Under Sleeper Pads

As further results for other boundary conditions (e.g. curved track) are also published in the UIC leaflet in the following additional results regarding initial quality, quality after tamping and expected service life will be discussed, based already on more than 60,000 cross sections compared. Fig. 6 shows typical track quality behaviour for a section of 54,000 gross tons per day and track, on the left side conventional ballasted track with rails 60E1 on conventional concrete sleepers, on the right hand side the same superstructure but with USP. The vertical red line shows the time of renewal, the green line a tamping action.

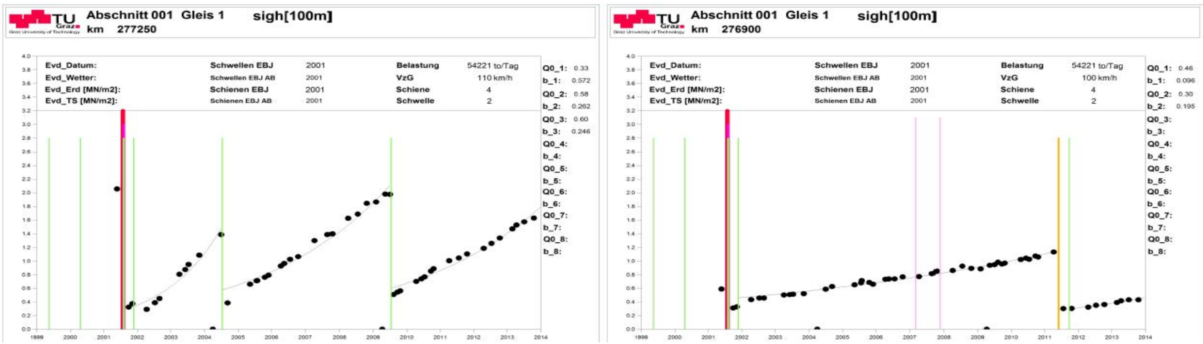

Fig. 6: Development of Track Quality

However, this is just a specific result. Therefore in Fig. 7 the quality after tamping and the deterioration rate is given for all checked sections. The quality deterioration rate is reduced from 0.14 to 0.07 as it was expected by the theoretical calculations. The quality after tamping is reduced from 0.5 mm standard deviation to 0.3 mm. As the analyses are based on Austrian data, all tamping actions were stabilised.

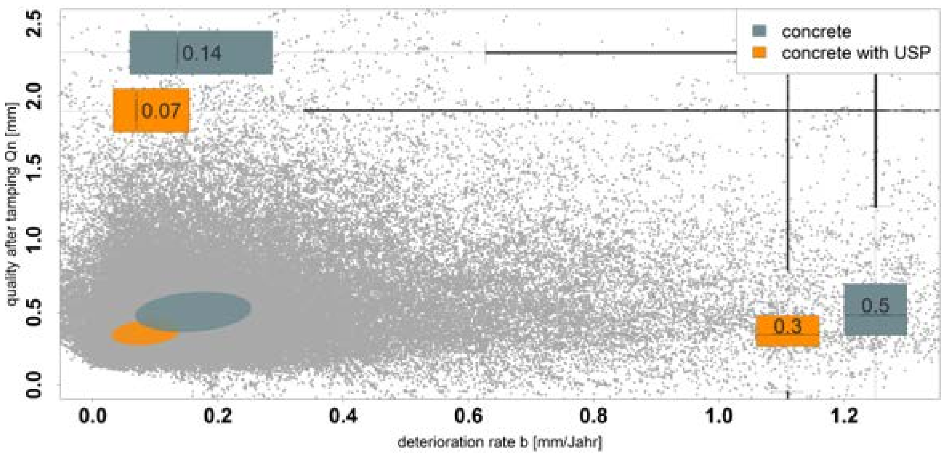

Fig. 7: Impact on Quality and Track Deterioration by Installing USP

These results underline the technical and consequently the economic efficiency of Under Sleeper Pads. Tab. 1 summarises the characteristics of wooden sleepers, concrete sleepers, and concrete sleepers with Under Sleeper Pads. The durability of the concrete sleepers stays the same, the lateral track resistance is increased as the ballast stones can press into the pads. Durability of track increases as the critical element of the ballasted track, the ballast, faces lower loads. In Austria concrete sleepers are the cheapest solution within investment, concrete sleepers with Under Sleeper Pads show the same price like wooden sleepers. However, cost of track renewal increase by 5 per cent, service life is prolonged by more than one third and maintenance can be reduced down to 50 per cent (average values for track on good sub-soil with proper dewatering systems). These facts result in an average reduction of total life cycle cost of one third.

Tab. 1: Characteristics of Ballasted Track with different Types of Sleepers

characteristics track	wooden	concrete	USP
durability sleeper	-	+	+
side resistance	-	+	++
contact area	+	-	++
durability track	-	+	++
investment	+	++	+
turnout	**wooden**	**concrete**	**USP**
same differences as for track and additionally			
stiffness	varying	varying	~ constant

5. VIBRATION ISOLATION WITH ELASTIC UNDER SLEEPER PADS

While USP are excellent for reducing maintenance costs, which has been already outlined in detail, they are also a cost-efficient and effective measure for reducing vibrations and secondary air-borne (refer to Fig. 8) noise next to ballasted track superstructure. Highly elastic pads offer a simple method for reducing vibrations on railway lines that is cost-effective in comparison with Under Ballast Mats (UBM). In addition, they exhibit all the positive properties of elastoplastic USP, such as increase of contact area and thus reduction of LCC.

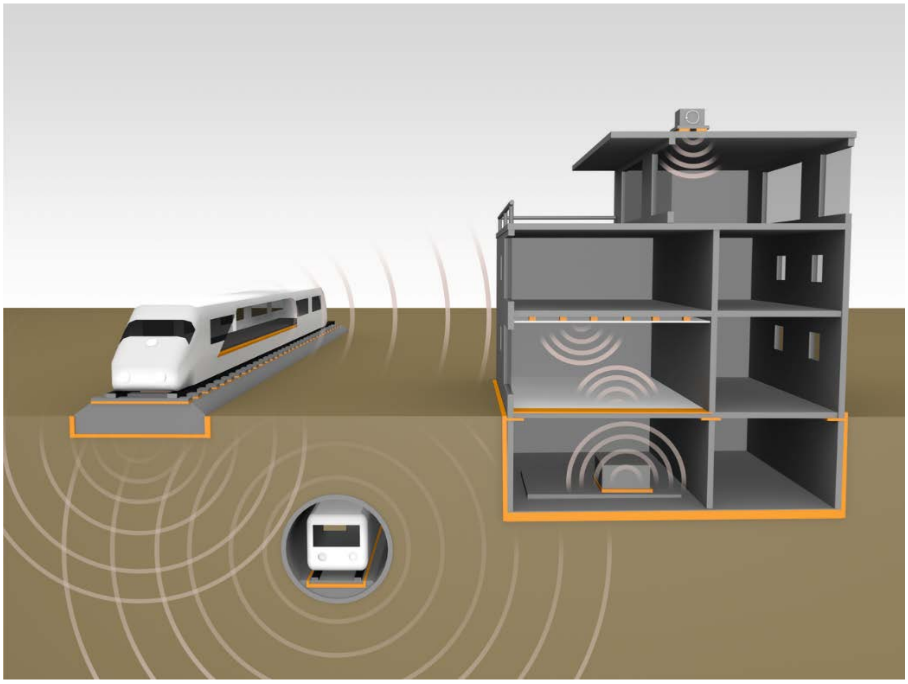

Fig. 8: Vibrations and structure-borne noise in the surroundings of railway lines

One advantage of USP over UBM is, that retrofitting can be executed a lot easier with USP, as sleepers can be exchanged one by one. Installation of UBM in an already existing track would be a lot more complex and costly. The vibration attenuation performance of UBM is of course higher, due to the full-surface decoupling, the bigger mass and the lower stiffness of the elastic layer. The decision between USP and UBM is a trade-off between total cost (product plus installation) and attenuation performance and has to be taken on the basis of project-specific boundary conditions.

Elastic sleeper pads with vibration-isolating characteristics are a very effective measure for reducing secondary air-borne noise. Since the vibration and sound emissions of the railway track are also significantly dependant on the quality of the superstructure, the more even the superstructure is, the lower the emissions are. In addition to this effect, the physical principle of vibration isolation plays a big part in the reduction of emissions. The performance of a vibration isolation solution is dependent on factors like overall mass, stiffness and system damping. By means of inserting an elastic element such as USP, an oscillating system is created. In the best case the eigenfrequency of the system is way below the spectrum of exciting frequencies that should be isolated, this is based on the principle of the one mass one spring single degree of freedom oscillator. The dynamic stiffness of the USP used must be tailored to the specific installation situation and the constraints of the project. Only if the technical solution is carefully engineered will the USP be able to exploit their full vibration isolation performance.

6. VALIDATION OF VIBRATION ISOLATION WITH ELASTIC ELEMENTS: MASS-SPRING SYSTEM IN BUDAPEST

Very high performing vibration attenuation systems can be found in slab track, where mass-spring systems with elastic polyurethane layers (full-surface, strip bearings or discrete bearings) greatly reduce the disturbance of adjacent buildings and residents.

6.1. Historical background

The Budapest tramway line dates back as far as 1866. When it opened, horses were still being used to pull what was then the first modern means of transport in Budapest, satisfying the need for mass transportation. As of 1887, the first sections of tramway lines were electrified. Since then the railway network has been extended to a track length of 147 km. The tramway line, the Budapest Metro network, the Budapesti Helyiérdekű Vasút suburban line and the city’s bus transport are all operated by the Budapest transport company BKV Zrt. (BKV for short).

Today’s transport operator is faced with more and more demands. On top of this, there is an increasing number of complaints from neighbouring residents, who are bothered by vibration and noise caused by the tramway line.

To counter this detrimental impact on their quality of life due to the railway traffic, BKV worked closely with Sika Hungária and Getzner Werkstoffe to create a test section with a mass-spring system (MSS). This protective measure is particularly useful for the properties that lie just a short distance from the centre of the track, as the cause is reduced directly at the source. The slab track with track slabs made of reinforced concrete and a full-surface bearing made of Sylomer® can also be installed quickly and accurately.

The aim of the project was to install a test section in the existing standard superstructure system and gather experience relating to renovation and operation. Noise and vibration studies and the response of neighbouring residents would add to the findings for the new MSS.

6.2. Test section and superstructure

The test section was approximately 96 m long and constructed on tramway line 52 which was built in 1980 on “Vörösmarty utca” street and is 7 km long. The line is operated with eight-axle type 6000 tramcars (which has an unloaded weight of 38.8 t and a capacity of 176 people each).

6.3. Implementation

The section had to be closed off during the renovation work, which took place in the first two weeks of November 2016. The already ageing superstructure had to be exposed along the length of the test section.

Above the 20 cm thick sub-base, a 5 cm thick asphalt layer encloses the top of the subgrade. Full-surface polyurethane bearings made of Sylomer® from Getzner with a thickness of 25 mm were installed on top of this. These take on the role of elastic bearings for the track slabs in relation to the substructure. A truck-mounted crane and a 4-rope crane sling were used to raise the pre-fabricated track slabs (6 m x 2.4 m x 0.18 m) onto the elastomer mats. The track slabs are separated by a gap of approximately 2 cm wide. (refer to Figs 9-12).

Two-stage transitions to the existing standard superstructure totalling 9 m and varying in stiffness were installed at the start and end of the test section. These homogeneously smooth the rail deflection when a tram passes over where the various superstructure components change and there is therefore an abrupt difference in the stiffness of the bedding. They thus reduce the superstructure's dynamic loads in these areas.

Fig. 9: First construction phase on the test section

Fig. 10: Transition area with Sylomer® mats

Fig. 11: Installing of the track slab

Fig. 12: Track slabs with rail channel

Sylomer® side mats with a thickness of 25 mm were used to laterally decouple the MSS and the adjacent road surfacing along the length.

Once the laying, alignment and welding work was complete, the renovation work was finished off by embedding the rails in the rail channels and filling the gap between and to the side of the track slabs with Sika Icosit KC 340/45 PU (Fig. 13).

Fig. 13: Completed track

6.4. The measurements

The effect the MSS had on vibration protection and reducing the secondary airborne noise was evaluated in two stages at separate times: the existing condition with the standard superstructure was evaluated in October 2016 and measurements were taken following installation of the MSS in November 2016.

Vibrations were recorded in a building directly next to the test section, selected by the Budapest transport companies, as a tram passed by. Data was acquired from two measuring points on the floor of the living area using geophones. At 3.6 m, the distance from the residential construction to the centre of track is very short. Measuring points were also set up at the same time on the pavement at a distance of approximately 3.4 m from the centre of track at three measuring sections, each with an 8-metre longitudinal clearance.

The secondary airborne noise of passing trams was also recorded simultaneously inside the building in question with the windows closed.

Fig. 14: Tram T6000 passage on the new MSS

6.5. Analysis and results

In order to evaluate the situation, 14 trams were recorded passing over the track section before and after the installation of the MSS. 1/3-octave-band velocity spectra were determined for each individual measurement and measuring point in a frequency range of 4 Hz to 400 Hz using the Max Hold method (“Fast time weighting”). For each measuring point, the energetic mean value of the 1/3-octave-band velocity spectra for all passing trams was calculated from the spectra of the individual passages. The arithmetic mean was calculated on the basis of two measuring points in the building / three measuring points on the pavement.

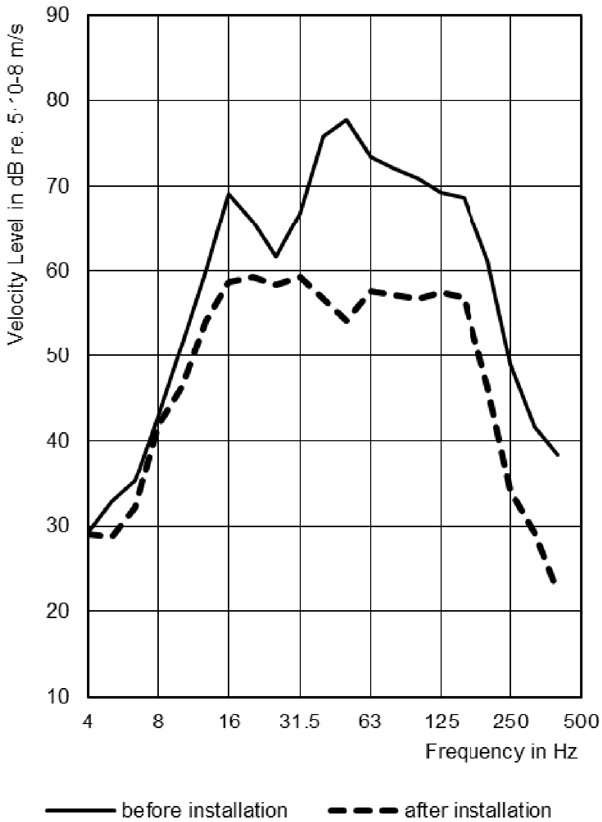

Fig. 15: 1/3-octave-band velocity spectra before/after renovation

Fig. 15 shows an example comparison of the measured 1/3-octave-band velocity spectra in the residential building before and after the renovation. The vibration emissions within the building were considerably reduced across a wide frequency range by the installation of the MSS.

These averages can be used to form a 1/3-octave-band velocity level difference for both groups of measuring points which can be presented on a graph as a function of the 1/3-octave-mid-frequency. This difference represents the vibration mitigating properties of the MSS in comparison to the standard superstructure and is termed insertion loss.

In locations where vibrations are generated by traffic loads, resulting in significant disruption to persons in buildings, the evaluation method set out in DIN 4150-2 is frequently applied and the maximum weighted vibration strength K_{BFmax} determined. If the average maximum weighted vibration strength K_{BFmax} is determined using the evaluation method as per DIN 4150 Part 2, this results in a value of 0.5 in the residential building prior to renovation, which can be described as “noticeable”.

After the renovation work, the average maximum weighted vibration strength K_{BFmax} was 0.09. This is below the value regarded as the "perception threshold" of $K_{BFmax} = 0.1$, below which there is, on average, no physical perception of vibrations. The measures reduced the perception of the vibrations in the residential building by approximately 80%. Comparable positive results in level reduction were achieved at the measuring points on the pavement.

It was possible to determine the maximum noise emission from the bogie passing by as an average A-weighted continuous sound level L_{AFeq}. Compared to the initial value of 57.3 dB(A), the maximum level in the living room was reduced by 9.3 dB to 48 dB(A).

The BKV are pleased that their trams now cause significantly less vibration and noise for the neighbouring residents on the renovated section. The residents in question confirmed that there is less disturbance from passing trams, which is even more meaningful than pure measurements.

7. SUMMARY

Under Sleeper Pads are a state-of-the-art technical solution reducing both Lifecycle Costs as well as reducing vibrations in railway tracks. Being already a standard product in countries like Austria, Germany, France and Italy, in recent years the positive effects of USP demonstrated also led to introduction of these elastic elements in the UK.

The biggest amount of all USP used across the globe is applied for LCC reduction reasons, a smaller part for vibration isolation purposes. This development can also be observed worldwide, as the adoption of USP into best practices for railway superstructure construction continues to spread across all continents.

To sum up all theoretical calculations and the practical experiences, it can be stated that only concrete sleeper with Under Sleeper Pads fulfil the demand for a more sustainable and cost effective superstructure.

In urban railway lines the need for vibration isolation is growing, as urbanization is progressing quickly. This need can be satisfied with high-performing mass-spring systems made from polyurethane. Getzner is the only manufacturer looking back on decades of proven long-term references that exhibit unaltered performance throughout the whole lifetime of the superstructure.

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HOMOGENISATION LAYER – A GREEN AND INNOVATIVE ANSWER FOR RECONSTRUCTION OF OLD CEMENT-CONCRETE ROADS

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SUMMARY

Article describes the procedure of pavement diagnostics of old cement-concrete roads, the approach to the solution for optimal pavement design based on results of diagnostic and engineering-geological survey; and the choice of building technologies which resulted in a new base layer conception named "Homogenisation Layer". The dominant criterion was also the minimising of construction time to reduce the impact of traffic restrictions. An important point of view was maximising (investigation of the technological limit) of utilization of original building materials while maximising of its qualitative degree and extent of re-use, and thus save the environment. Finally, the task is also to save money and energy.

The specific boundary conditions for solution of the task were: questioned parameters of the pavement subgrade and base courses built about 50 years ago, classification of the water regime and in particular a very short time for providing the solution.

One of the most critical conditions of the original pavement construction was the serious inhomogeneity of both the base course and the subgrade, mostly caused by unmaintained joints and cracks. The described solution meanwhile grew as a routine response to such a case in Slovakia.

1. INTRODUCTION

Cement-concrete pavements built in seventies of the last century still serve the designed purpose, although in limited extent. These pavements are far behind the designed lifetime. Their technical conditions request reconstruction, mostly from the reason to ensure driving comfort and because of high number of local defects and because of parallel acceleration of degradation processes caused by neglecting and impossibility of maintenance. This article shows a procedure of pavement diagnostic, approach to the pavement design based on results of diagnostic and engineering geological survey, and selection of used technology from aspects of minimization construction time to lower the impact of traffic restrictions, not only maximizing usage of an original construction material, but also maximizing quality level, reusing rate and by that the environmental protection and last but not least also aspects of saving money and energy.

2. INTRODUCTION OF THE TOPIC

The issue of reconstruction 40 – 50 years old cement-concrete pavements brings us questions about mechanical efficiency of whole pavement construction at several levels. Possibly the biggest question is the condition of subgrade. Its diagnostic in the view of bearing capacity is possible by means of e.g. FWD deflectometer.

Fig. 1: Illustrative view on the degree of deterioration of cement-concrete road I/65. Previous successful local repairing is visible on the right side and serves a comparison with the stage before.

This is usable without troubles only in the intact (unbroken) cement-concrete plate fragment. However, when we need to know its condition (parameter of bearing capacity) under a long term neglected transversal joint, our chances to evaluate it by using non-destructive method are seriously limited. It can be noted, that the subgrade can be divided in places under the pavement fragment and under the places deteriorated by joints and cracks. In addition, the bearing capacity parameter of the subgrade in pavement fragment is equivalent to the level of technical abilities of compaction technique of the seventies.

Another level of questions is hidden in the issue of solving the base course. That depends on approach to designing a wearing course. According to the state of deterioration of mentioned pavements, a repairing of the existing cement-concrete plates was irrelevant. Moreover, the investor decision was to design an asphalt pavement.

TPA Ltd handled in 2014 a diagnostic engineering geological and hydrological survey, bearing capacity diagnose and design of reconstruction of three important road sections with "old" un-dowelled cement-concrete wearing course (Boros, 2014b; Boros, 2017a; Boros, 2017b). In 2015 the reconstruction of the first section – road I/62 and in 2016 reconstruction of the second section – road I/75 was executed. When the reconstruction was finished, our company executed additional FWD measurements, which allowed us to make comparison between parameters of the base course in the design process and parameters from the existing pavement. Another FWD measurements were made in 2018 at both sections I/62 and I/75 and at road I/65.

3. DIAGNOSTIC OF BEARING CAPACITY BEFORE RECONSTRUCTION

Due to the bearing capacity investigation of road sections intended for reconstruction were made standard non-destructive measurements of the deflection bowl from approximately the same load by means of FWD.

Fig. 2: View on the loading plate and sensors arrangement of the used FWD apparatus type Dynatest 8000 during the measurements on the road I/62 before reconstruction

These measurements are based for theoretical calculations of the half-space modulus of the pavement (surface moduli), elastic layer moduli and design of the pavement reconstruction, respectively pavement strengthening. Concerning the measurements on the cement concrete plates and focusing on the subgrade bearing capacity investigation were for the loading plate diameter of 300 mm chosen the load value of 75 kN.

The measurements were executed on the fragment of the cement concrete plates with effort to measure on the unbroken surface. No measurements were executed in the places of corners and edges, because investigation of load transfer between the cement-concrete plates was not the subject of diagnostics (because the actual road condition suggested as impossible to remain the original plates).

The thickness of pavement construction identified from drilled cores varies between each other. Different complementary information about the condition of the base course and subgrade were obtained from the documentation about earlier local reconstructions in the past. The thicknesses of the cement-concrete wearing course differed from 230 mm to 280 mm. Beneath of cement concrete plates occurs in different sections either asphalt layer or cement bounded base course on gravel sand but also asphalt layer on cement bounded base course on gravel sand. The subgrade of the pavements is formed by fine-grained cohesive soil type of loess and by non-cohesive soils type fluvial sand gravel sand. Bearing capacity of the subgrade with natural moisture content is in the least favourable places characterized by CBR ration of 5 %. For security reasons (possible deviations from the natural moisture) in the calculation was considered the CBR value of 3 %.

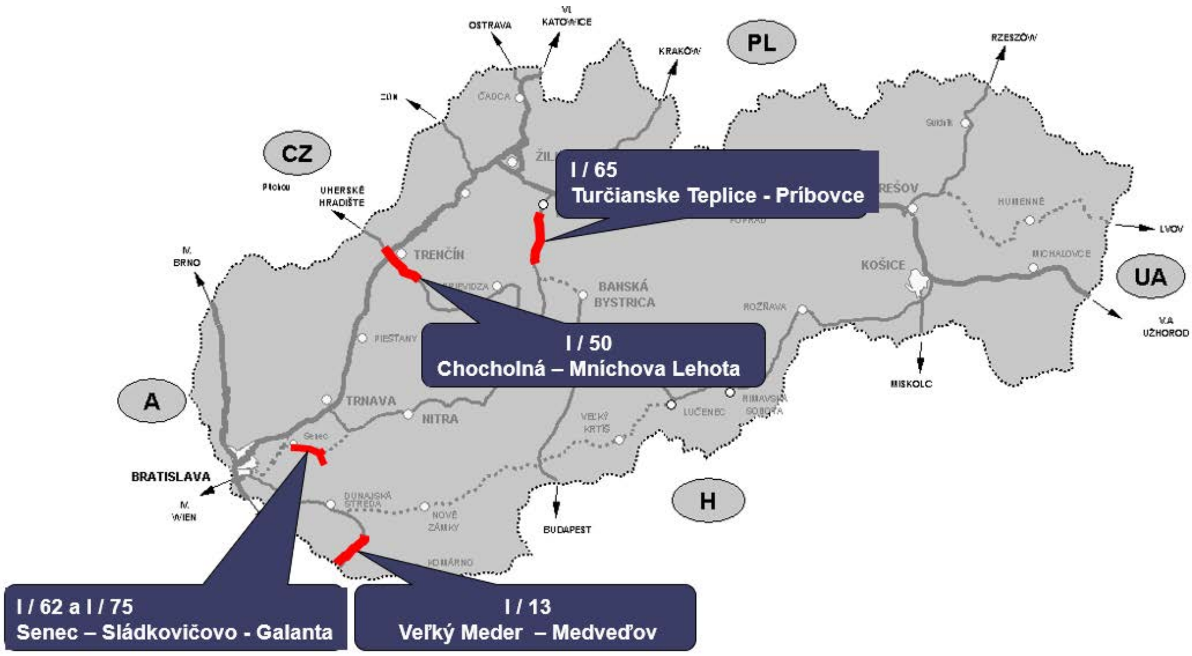


Fig. 3: Localization of road sections with cement-concrete wearing course for reconstruction

Pavement surface moduli (obtained from FWD) of the investigated sections (in the plate fragment but in random places depending on measuring step) vary between 3000 and 230 MPa. The elastic moduli of the unbroken cement concrete plates are in average about 30000 MPa.

4. DISCUSSION ABOUT THE PAVEMENT BEARING CAPACITY RESULTS

Considerations and conclusions on measurement results were first formulated for the first diagnosed section of I/62 and I/75; From the results of measurements and calculations of investigated sections is obvious, that the road in the place of cement-concrete plate fragment has in average high values of surface modulus.

(For example: The maximal value of Surface moduli (E0 MPa) of road construction of the road I/62 is 3020 MPa and minimum value is 229 MPa, average value is 1740 MPa, standard deviation is 437 MPa).

This confirmed the expectations before measurement. According to the deteriorated condition of local places (broken plate, pot-hole repairs by using asphalt concrete etc.), there also appeared some extreme low values of surface modulus. This confirmed a huge difference in bearing capacity of mentioned sections. The lowest measured values of surface modulus in the range of 200 MPa – 300 MPa are inconvenient even for a flexible pavement of the lowest traffic load category IV. – VI. (according to technical conditions TP 01/2009, sufficient value for this category is at least 400 MPa). The other related problem, even more serious, is the deterioration level of base and subgrade in the places of long-time neglected transversal joints.

It was needed to consider high level of inhomogeneity in the bearing capacity of diagnosed sections at the design process of reconstruction. Two possibilities are to disposal for solving the reconstruction: detailed detection of damaged places and repair them before reconstruction or using the line technology of cold recycling in place and by that create a homogeneous base course.

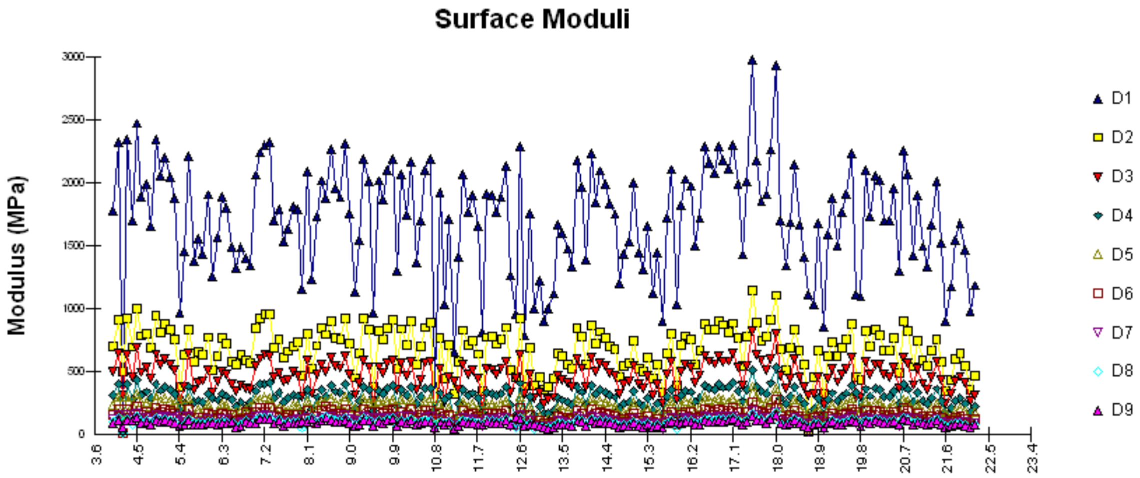


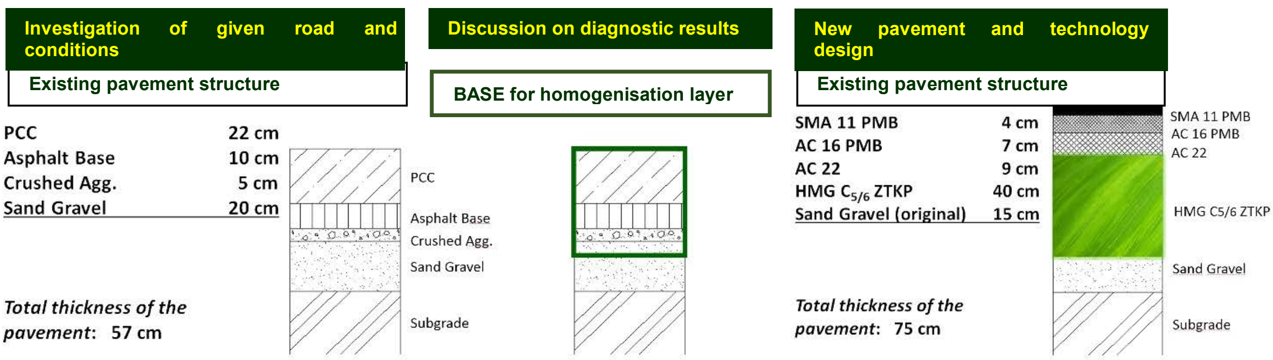
Fig. 4: Example of Surface moduli (E_0 MPa) calculated from the measurements made by the deflectometer FWD at the loading 75 kN in the section of the road I/62 km 4,00 – 22,15, right side

5. PAVEMENT DESIGN AND PLANNING OF THE BUILDING TECHNOLOGY

One of the investor's initial requirements was the pavement design of an asphalt pavement for design period of 20 years.

All border conditions defined in Slovakian pavement design method anchored in the document TP 3/2009 (traffic loading, geomorphological data, climate conditions, hydrogeological conditions, subgrade bearing capacity) were considered in the design process.

Tab. 1: The design process of the Homogenisation Layer



Engineering aim was to design the pavement reconstruction to be able to remain the original subgrade. This is possible to do by more ways. Decision fell on maximizing of evaluation of local material sources to ensure highest possible operational performance of reconstructed pavement. The pavement design solution focused on ensuring the homogeneity (degraded places were eliminated by using hydraulic bounded subbase course) was developed at the first section (I/62 and I/75) and was applied on two other projects (I/65 and I/50). As result was the hydraulically bounded "homogenisation" base course in the thickness of 40 cm. Searching for fragmentation technology of existing cement-concrete plates to the suitable fraction, which allows to add the hydraulic road binder and mill together with the part of existing base course using milling machine in the thickness of 40 cm was a big challenge "behind the desk", because the lack of experiences.

Tab. 2: Pavement structure (simplified, without definition of the tack coat, type of used bitumen etc.)

Pavement layer	Layer thickness [cm]
SMA 11 PMB	4
Asphalt concrete binder course ACL 16 PMB	7
Asphalt concrete base course ACP 22	9
Homogenisation layer C5/6 (HMG)	40
Gravel sand (original)	15
Total thickness of pavement:	75

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6. TRIAL SECTION FOR VERIFICATION OF THE BUILDING TECHNOLOGY

Fig. 5: a) View on the resonance head of the RMI RB 500 apparatus during the fragmentation of cement concrete plates; b) The result on the road after passing of the machine

Due to the lucky fact, that STRABAG company realized in 2015 on the road I/75 Galanta bypass, which part was also the removing a part of the original section of this cement concrete road, it was able to verify the designed reusing technology (fragmentation and milling) in real boundary conditions (before starting the reconstruction).

Fig. 6: a) View on the road building material originate from the cement-concrete plates and b) partially from base course after fragmentation and milling

The section was fragmented by using RMI RB 500 machine and after that milled by soil stabilizer Wirtgen WR 2500. The aim of this trial was to test the availability of designed technology for assigned purpose. The resulted fragmented material from cement concrete plates mixed with the present base course by milling with road stabilizer was tested in the laboratory. The resulted unbound material had suitable grain-size distribution and the usage of proposed technology was confirmed.

7. CHARACTERISTICS OF THE BASE LAYER

Based on road diagnostic there was a need to design so called homogenization layer (because the aim was to remain the existing subgrade) for the new pavement construction. The abbreviation of "Homogeneous Layer" as "HMG" was first called by the planning engineer Mr. Brliť. Parameters for HMG in design process were considered as for CBGM C5/6. For the base course HMG were elaborated special technical-qualitative conditions (ZTKP), because HMG layer is no a standardized layer. HMG layer originates from homogenization of R-material created after fragmentation of cement-concrete wearing course and other materials of existing base course, which are situated in the depth of 400 mm measured from the existing roadway surface. Homogenization is achieved by mixing of obtained R-material and potential crushed aggregate fraction 32 mm by powerful road milling machine with mixed, slow-hardening bonding material. The correctness of the design is confirmed by the 3 year exploitation of road I/62 and I/75 between Senec and Galanta.

8. RESULTS OF FWD MEASUREMENTS AFTER RECONSTRUCTION OF THE PAVEMENT

After putting the road section I/62 (km 9,344 – 22,146) into service, there were executed deflection measurements by two deflectometers Dynatest 8000 type (TPA) and KUAB type (SSC Slovak Road Administration). According to a date of measurement (2nd December 2015), the boundary conditions were: average air temperature 6 °C, average roadway surface temperature 7 °C, load 50 kN. Based on analyse of measurement results from SSC by evaluation program CANUV was determined that minimal residual lifetime of 20 years was achieved in every measured place.

Fig. 7: View on the road I/62 after the reconstruction and the measuring equipment FWD Dynatest 8000; December 2015

The results of layer elastic moduli obtained from back-calculations are as follows:

Model description: *E1* – asphalt packet 20 cm, *E2* – base course HMG 40 cm, *E3* – subgrade

Tab. 3: Left side, measuring points: 60, back-calculation method: FEM

Layer	**Average value [MPa]**	**Standard deviation**
E1	18 648	1.220
E2	2 852	1.805
E3	209	1.173

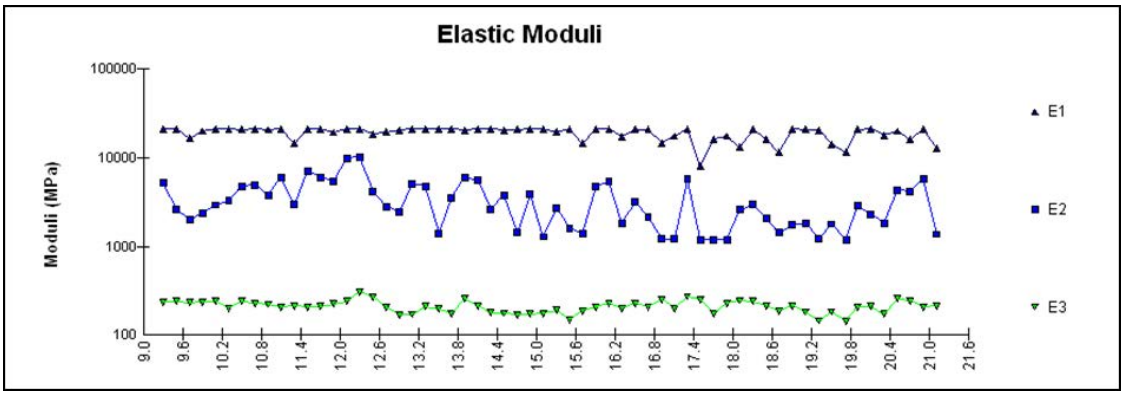

Fig. 8: Layer elastic moduli in the pavement model (I/62 left side) obtained by back-calculation from measured deflections using FWD Dynatest 8000 (2nd December 2015; average air temperature 6 °C, average pavement surface temperature 7 °C, load 50 kN)

Tab. 4: Right side, measuring points: 59, back-calculation method: FEM

Layer	Average value [MPa]	Standard deviation
E1	18 247	1.284
E2	2 652	1.891
E3	208	1.227

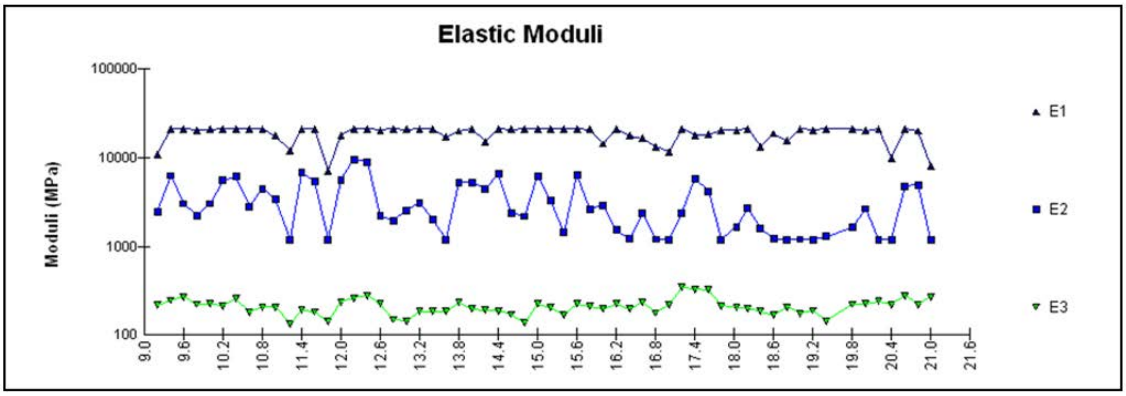

Fig. 9: Layer elastic moduli in the pavement model (I/62 right side) obtained by back-calculation from measured deflections using FWD Dynatest 8000 (2nd December 2015; average air temperature 6 °C, average pavement surface temperature 7°C, load 50 kN)

9. RESULTS OF FWD MEASUREMENTS 3 YEARS AFTER RECONSTRUCTION OF THE PAVEMENT

Approximately 3 years after the I/62 (km 9,344 - 22,146) road section was put into operation, the FWD measurement by deflectometer Dynatest 8000 was repeatedly measured. On 14.11.2018, the boundary conditions were as follows: average air temperature 14 °C, average pavement surface temperature 12 °C, load 50 kN.

The results of layer elastic moduli obtained from back-calculations are as follows:

Model description: *E1* – asphalt packet 20 cm, *E2* – base course HMG 40 cm, *E3* – subgrade

Tab. 5: Left side, measuring points: 64, back-calculation method: FEM

Layer	Average value [MPa]	Standard deviation
E1	16 073	1.116
E2	4 665	1.377
E3	279	1.281

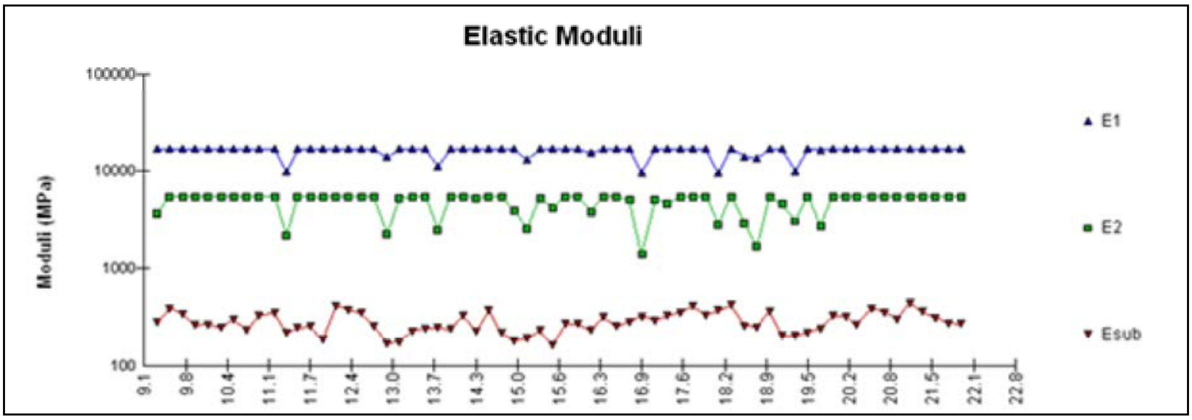

Fig. 10: Layer elastic moduli in the pavement model (I/62 left side) obtained by back-calculation from measured deflections using FWD Dynatest 8000 (14th November 2018; average air temperature 14 °C, average pavement surface temperature 12 °C, load 50 kN)

Tab. 6: Right side, measuring points: 63, back-calculation method: FEM

Layer	Average value [MPa]	Standard deviation
E1	16 353	1.115
E2	4 442	1.593
E3	292	1.265

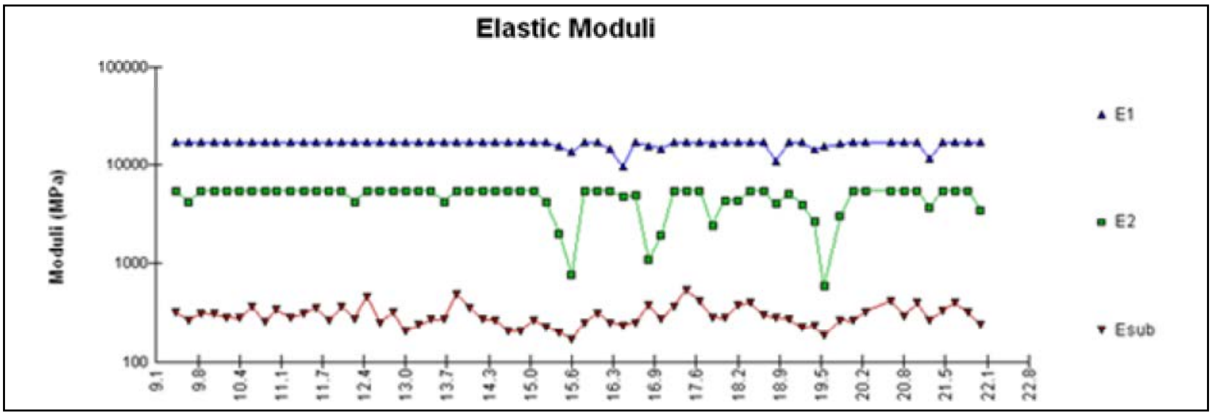

Fig. 11: Layer elastic moduli in the pavement model (I/62 right side) obtained by back-calculation from measured deflections using FWD Dynatest 8000 (14th November 2018; average air temperature 14 °C, average pavement surface temperature 11 °C, load 50 kN)

10. CONCLUSIONS

The "Homogenisation layer" is an extra strong hydraulically bound base layer (layer thickness 40 cm) by using cold recycling in situ and reuse of existing old concrete and underlying base course.

An important fact is that this layer can be built using recycling technology, which means it's not just the idea of the office table.

This homogenization layer compensates for imperfections in the carrying capacity of existing (old) roads in a linear, rapid technological step. Even the homogenization is not only in the longitudinal area but also in the wide area (roadside).

The innovative solution is the (today's) "maximum" possible layer thickness 40 cm (at high quality), which considerably extends the service life, saves on transport material costs and maintains old base courses and the subgrade.

From the modulus of elasticity of the pavement layers point of view, which are obtained by the back-calculation of the deflections measured by the FWD immediately after the reconstruction, their values are, compared to laboratory measurements (under similar conditions: temperature of 5 °C, and frequency of approximately 20 Hz - with a pulse duration of 25 ms) very high (18000 MPa). This is due to the low measurement temperature, but also due to the fact, that asphalt layers in situ are loaded by different way that in a laboratory condition - in particular, the spatial loading pattern of the actual pavement versus the linear loading pattern of the sample in the laboratory. According to the authors opinion, the loading by the FWD device is closer to reality than loading of the sample in the laboratory. It is apparent from repeated measurement after 3 years of exploitation that the asphalt stiffness modules have unusually high values (16000 MPa) even at 14 °C.

The benefits we can summarized in these 6 "WIN" cases:

ENVIRONMENT: Keeping original layers of pavement base and subgrade (saving natural resources)
RENEWING: Homogenization of bearing capacity of underlying pavement system (unification and adaptation to new requirements)
COST SAVING: Reducing the energy consumption of building construction (lowering prices)
TIME: Choice of technology with smooth and continual character (time of construction)
RESOURCE UTILIZATION: Maximizing the technological reach of the construction equipment and thus more efficient use of resources
LONG LIFE: Extended pavement life.

Fig. 12: I/62 Senec – Sládkovičovo road – FWD measurement by Dynatest 8000, 3 years after reconstruction; November 2018

Fig. 13: I/65 Turčianske Teplice – Príbovce road – FWD measurement by Dynatest 8000 "fresh" after reconstruction; November 2018

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SERVICE LIFE DESIGN FOR TRAFFIC INFRASTRUCTURE OF CONCRETE – STATE OF THE ART AND NEW APPROACHES

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SUMMARY

The performance and durability of concrete structures has to be guaranteed by means of an appropriate design approach, such that the target service life may be achieved without substantial maintenance measures. The currently used descriptive concept for durability design does often fail for various reasons. The probabilistic design approach, also termed performance concept, is clearly superior but also more sophisticated, as it needs appropriate degradation models. While a few reliable models exist for specific degradation processes, there are still considerable gaps in our knowledge with respect to combined deterioration mechanisms and the interaction of degradation processes, which mostly prevail in practice. This paper indicates the application of the probabilistic design approach for concrete structures and addresses as well the consideration of combined effects and material degradation interactions.

1. INTRODUCTION

The procedure for durability design as given in the *fib* Model Code for Service Life Design (*fib* Bulletin 34, 2006) is focused on single deterioration mechanisms of concrete structures, which are treated individually. While the knowledge of single deterioration mechanisms at individual exposures conditions is well developed, there is a substantial lack of understanding the effects of combined deterioration mechanisms, which occur in practice. Usually, concrete structures and their structural components are exposed to different environmental impacts, which lead to a simultaneous occurrence of different deterioration mechanisms. Moreover, these different deterioration mechanisms may interact with each other. The identified interactions between the individual deterioration mechanisms have to be analyzed and modelled in detail in order to modify the given deterioration time models properly.

Against this background, a new approach for the durability design was developed, considering combined deterioration mechanisms and their interactions. This approach involves time-variant deterioration models concerning the durability related degradation mechanisms (e.g. carbonation induced corrosion) as well as singular structural risks (e.g. cracks or leakage). In the course of the reliability analysis the interaction of the combined deterioration mechanisms – here exemplarily shown for chloride and carbonation induced corrosion – is taken into account by implementing a scaling factor, which represents the modification of the material behaviour as a result of the considered interaction.

The carried out computer based investigations have proven the practicability of the new approach for the durability design. By means of the method indicated in this paper, a first step is made towards an interaction model that is user-friendly, flexible and gives the possibility to be extended. The level of detailing related to the modification of the used deterioration-time models can be increased if necessary. Furthermore, the service life prediction can also be extended by taking into consideration the effects of singular failure risks caused by structural safety problems.

2. CONCEPTS OF SERVICE LIFE DESIGN

Subsequently both concepts, the descriptive concept and the performance concept of service life design of concrete structures, will be briefly outlined, and the essential differences between both concepts will be pointed out.

2.1 The descriptive concept

The existing procedure for durability design of civil concrete structures is based on the empirical experience related to the material and structural behaviour. The national and international standards imply special descriptive limits (for resistances) in connection with rough classifications of environmental conditions (for actions) in order to ensure the durability of structures for an approximate defined minimum lifetime, e. g. 50 years according to valid standards (DIN EN 1990, 2010; DIN EN 206-1, 2001; DIN 1045-2, 2008). For instance, limiting values of the concrete composition (e.g. water/cement ratio) and properties (e. g. concrete strength) were defined to protect the concrete and the reinforcement from damaging effects resulting from environmental exposures, e. g. frost attack or chloride ingress. Therefore, this concept is a prescriptive approach that considers different environmental actions on the components of concrete structures and the material resistance in a descriptive way (see Fig. 1, left).

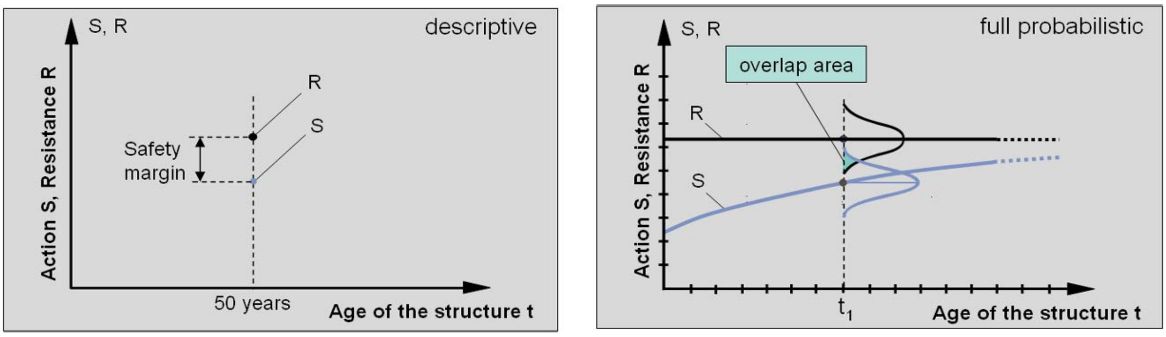

Fig. 1: Schematic illustrations of the descriptive concept (left) and the performance concept given in a full probabilistic version (right)

The descriptive concept is connected with several unfavourable consequences allowing only a rough estimation of the durability. Neither the relevant deterioration mechanisms nor the actual material resistance were considered in a realistic way. Furthermore, the effective safety margin between action and resistance is unknown to the designer. In addition, it is not possible to quantify the durability related concrete properties for a defined service life (e.g. 20 or 80 years).

2.2 The performance concept

In contrast to the descriptive concept, the performance concept allows the determination of the durability of concrete structures in a quantitative way (see Fig. 1, right). Hereby, the increasing damage process with time, i. e. the interaction of action and resistance, affecting the concrete structure is modelled by means of appropriate deterioration-time laws, and the material resistance is additionally quantified. Since there are several uncertainties in the action-and resistance-related parameters, it is necessary that their variability has to be described by means of statistical parameters. Consequently, the safety margin between the well-defined functions for the action S and the resistance R can be expressed in terms of the failure probability, see the overlap area between the two curves in Fig. 1, right.

The performance concept allows the analysis of the time-dependent increase of damage and the failure probability according to a defined unintended condition – the limit state – of the structure. The decisive advantage of the performance concept is based on the fact that the time-dependent durability of concrete structures can be expressed in terms of the failure probability or reliability indices, respectively.

It is to be expected that the next generation of European standards will include probabilistic methods for durability design at least to some extent. The required knowledge and tools have been developed within recent years (Sarja, Vesikari, 1996; Sentler, 1983; The European Union – Brite EuRam III, 1997; The European Union – Brite EuRam III, 1998; The European Union – Brite EuRam III, 1999; The European Union – Brite EuRam III, 2000). For instance, well-established models, which describe single degradation processes in uncracked concrete for the initiation phase, are already given in the *fib* Model Code for Service Life Design (*fib* Bulletin 34, 2006). Further approaches are under discussion.

By means of the available statistical tools and given degradation-time laws the prediction of the lifetime of a structure is feasible for civil engineers in practice. The individual steps of the procedure of a performance-based durability design can be taken from the literature (Gehlen, Mayer, Greve-Dierfeld, 2011). In the following chapters the focus will be put on the practical applications of service life design.

3. SERVICE LIFE DESIGN – DESIGN STEPS

In the subsequent paragraphs, the essential elements and design steps for the service life design of civil concrete structures are briefly summarized.

3.1 The deterioration processes

The increasing deterioration with time, i. e. the gradual loss of durability due to environmental actions on concrete, has to be described by means of deterioration-time laws, also called material laws or material models. Such laws should preferably take into consideration real physical or chemical mechanisms. For instance, a typical degradation process is the carbonation-induced corrosion of the reinforcement. Considering this process in terms of action S and resistance R, the action is described by means of the material law for the carbonation progress into the concrete taking environmental and material parameters into account. The resistance is for example given by the thickness of the concrete cover.

3.2 The model parameters

The parameters included in the material models for the action S and the resistance R are not exact values because they scatter around average values, see Fig. 1 (right). In practice, this can be easily observed for the carbonation depth (action S) as well as for the concrete cover (resistance R) in a concrete member. Hence, the varying parameters are considered as random variables, also called basic variables. If such a basic variable is measured, the corresponding mean value and coefficient of variation as well as the type of the distribution function have to be determined.

3.3 The limit states

A limit state is defined as a condition at which a civil structure or a structural component ends to perform its intended function. In the case of carbonation induced corrosion of the reinforcement a limit state is fulfilled when the carbonation front reaches the reinforcement. Correspondingly, for chloride induced corrosion a limit state is reached if the actual chloride content is equal to the critical chloride content in the depth of the reinforcement.

3.4 The service life of concrete structures

The loss of durability, i. e. the increase of the deterioration with time, reduces the reliability of a concrete structure. In order to evaluate this reliability at any age of the structure, a reference period for the service life has to be defined. Reference values of the service life of buildings and structures are listed in relevant standards and guidelines. As an example, the intended service life of residential buildings and other simple engineering structures is 50 years, for complex engineering structures it is often 100 years (DIN EN 1990, 2010), or even more.

3.5 The failure probability and the limit state function

The failure probability p_f is defined as the probability for exceeding a limit state within a defined reference time. When this occurs an unintentional condition of a considered building component is reached (Hurtado, 2004). The magnitude of the failure probability is closely connected with the interaction of the resistance and the action functions and varies with time, see Fig. 1 (right). This interaction – here for a fixed point in time – may be described by means of the so-called limit state function Z which is defined according to Eq. 1.

$$Z = R - S \tag{1}$$

The function Z represents the elementary form of a limit state function in which R and S are random variables. If the value of Z turns to zero, the limit state will be reached. The statistical properties of the function Z can be expressed in the form of a distribution function, if this function is considered to be normal distributed and the resistance R as well as the action S are expressed using related mean values μ and standard deviations σ.

By means of the introduction of the so-called reliability index β, a direct correlation between this reliability index β and the failure probability p_f is obtained. In case of a normal distributed limit state function Z, the failure probability p_f can be determined directly by Eq. 2.

$$p_f = p\{Z < 0\} = \Phi(-\beta) \tag{2}$$

In Eq. 2 $\Phi(.)$ is the standard normal distribution function (Melchers, 2002). The correlation between various values for the failure probability p_f and the reliability index β is shown in Tab. 1.

Tab. 1: Values for the failure probability p_f and the related reliability index β (DIN EN 1990, 2010)

p_f	10^{-1}	10^{-2}	10^{-3}	10^{-4}	10^{-5}	10^{-6}	10^{-7}
β	1.28	2.32	3.09	3.72	4.27	4.75	5.20

The calculation of the failure probability p_f for a building component considering a particular mechanism related to durability (e.g. carbonation induced corrosion of the reinforcement) may be performed by the use of the subsequent Eq. 3.

$$p_f = p\{Z < 0\} \leq p_{target} \quad (3)$$

Finally, the calculated failure probability p_f has to be compared with the target failure probability or the target reliability index β, respectively. The target values can be taken from national and international standards or will be defined by the owner of the concrete structure.

3.6 The target reliability index

The target values of the reliability index β_{target} depend on the consequences of failure (loss of serviceability) and the relative cost of safety measures. Tab. 2 indicates target values of the reliability index β for building components in the serviceability limit state (SLS), see JCSS, 2001 and Rackwitz, 1999.

Tab. 2: Target values of the reliability index β depending on the relative cost for safety measures

Relative cost for safety measures	**Reliability index β (JCSS, 2001)**	**Reliability index β (Rackwitz, 1999)**
High	1.3 ($p_f \approx$ 10 %)	1.0 ($p_f \approx$ 16 %)
Moderate	1.7 ($p_f \approx$ 5 %)	1.5 ($p_f \approx$ 7 %)
Low	2.3 ($p_f \approx$ 1 %)	2.0 ($p_f \approx$ 2 %)

Considering the case of depassivation of the reinforcement due to carbonation or chloride ingress the target reliability index is recommended to be β = 1.3 by the *fib* Model Code of Service Life Design (*fib* Bulletin 34, 2006).

3.7 System reliability

In the previous section the procedure for service life design was shown only for structural components considering a single limit state. However, one has to keep in mind that typical concrete structures are complex systems. In general, they are composed of numerous structural components which have to satisfy more than one limit state criterion according to the different environmental exposures that stress the structure simultaneously. Therefore, it is necessary to distinguish between the reliability of components and the reliability of systems.

In view of a system reliability analysis there are two basic elementary systems: the series system – called also "weakest link system" – and the parallel system – called also "redundant system". A series system fails if any of the system elements fail, and a parallel system fails definitively if all elements fail. By means of mathematical rules one can define the lower and upper bounds of the failure probability of the system (Melchers, 2002).

The simple bounds for the failure probability of a series system can be calculated by means of Eq. 4:

$$\max\left[p_{fi}\right] \leq p_{f,series} \leq 1-\prod_{i=1}^{n}\left(1-p_{fi}\right) \tag{4}$$

The simple bounds for the failure probability of a parallel system can be calculated using Eq. 5:

$$\prod_{i=1}^{n} p_{fi} \leq p_{f,parallel} \leq \min\left[p_{fi}\right] \tag{5}$$

The bounds for the failure probability of civil structures depend on the statistical dependences of the identified failure events. Examples for probabilistic service life design of complex concrete structures are given in Müller, Vogel, 2011.

4. COMBINED DETERIORATION MECHANISMS

In the current practice, durability-related exposures (e. g. carbonation, chloride or frost attack) and singular risks (e. g. cracks and leakage) are often treated separately. However, in practice these exposures at concrete structures occur usually in a combined manner (Holt, Kousa, Leivo, Vesikari, 2009). Therefore, it must be identified which exposures act to which component (e. g. columns, slabs, groundwork) of the structure.

In order to set limits related to the above described problem a simple concrete bridge superstructure with limited durability related exposures is considered to show exemplarily the basic working steps related to the identification of combined deterioration mechanisms (see Fig. 2 and Tab. 3).

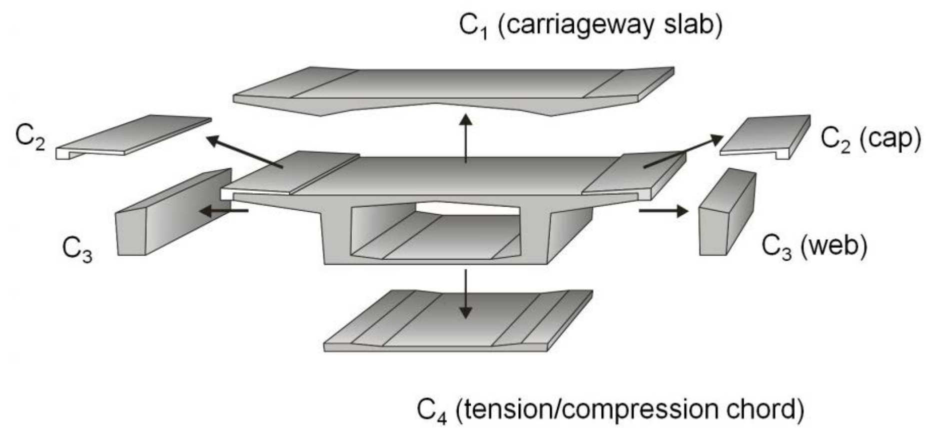

Fig. 2: Schematic illustration of a component subdivision of a bridge superstructure (box girder bridge)

In a first step it is necessary to identify the main components of the bridge superstructure. Subsequently, the superstructure has to be subdivided into its main components. Fig. 2 shows the principle of a component subdivision. Here, the main components are the carriageway slab (C_1), the caps (C_2), the webs (C_3) and the tension/compression chord (C_4).

Tab. 3: Structural components of a bridge superstructure and their possible exposures

Component	Denotation	Durability related exposure	Degradation process
C_1	carriageway slab	- carbonation - chloride ingress	- carbonation induced corrosion of the reinforcement - chloride induced corrosion of the reinforcement
C_2	caps	- carbonation - chloride ingress - freeze/thaw attack	- carbonation induced corrosion of the reinforcement - chloride induced corrosion of the reinforcement - internal cracking and/or scaling of the concrete
C_3	webs	- carbonation - chloride ingress - freeze/thaw attack	- carbonation induced corrosion of the reinforcement - chloride induced corrosion of the reinforcement - internal cracking and/or scaling of the concrete
C_4	tension/ compression chord	- carbonation - chloride ingress	- carbonation induced corrosion of the reinforcement - chloride induced corrosion of the reinforcement

For the component subdivision the appropriate level of detail depends on the given structure itself. It is important to classify the different components according to their function. In a further step every component of the superstructure has to be assigned to the different exposure conditions and degradation mechanisms, which were identified at the individual components of the bridge superstructure (see Tab. 3).

Tab. 3 contains some examples for a reasonable classification of the durability related exposures (carbonation and chloride induced corrosion and freeze/thaw attack) with reference to the corresponding structural components (Brühwiler, Menn, 2003; DIN-Fachbericht 100, 2010). Other durability related exposures were not considered here for reasons of simplification.

5. PROBABILISTIC SERVICE LIFE PREDICTION

5.1 Single deterioration mechanism

The failure probability of the components carriageway slab, webs and tension/compression chord (see Fig. 2) of the bridge superstructure is determined considering only the main durability related exposure for each component. Fig. 3 visualizes the relevant exposure conditions for the individual components of the bridge superstructure.

The chloride induced corrosion of the reinforcement (E_1) is related to the carriageway slab (component C_1), the freeze/thaw attack (E_2) is related to the webs (component C_3) and the carbonation (E_3) is related to the tension/compression chord (component C_4).

For the exemplarily probabilistic service life prediction the target reliability index β is set to be 1.7, and the considered lifetime is 80 years. For this example, the corresponding deterioration-time laws have been taken from the literature (*fib* Bulletin 34, 2006; Sentler, 1983). The magnitude of the required parameters and their statistical characteristics have been taken or derived from The European Union – Brite EuRam III, 1998 and The European Union – Brite EuRam III, 2000.

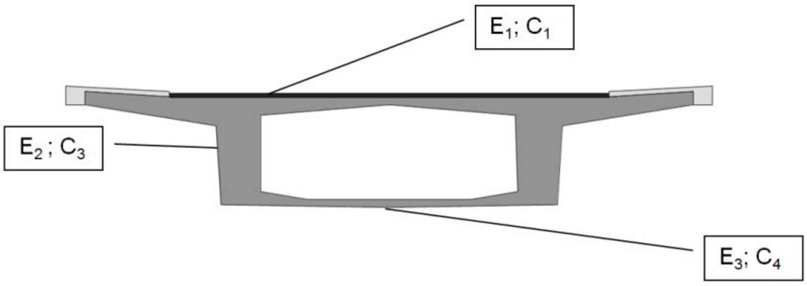

Fig. 3: Bridge superstructure with its components and corresponding relevant exposure conditions

Tab. 4 shows the obtained results of the reliability analysis of the components of the superstructure. This analysis was performed applying the software STRUREL (RCP GmbH, 2003). If it is assumed that the most severe exposure – here the chloride induced corrosion – controls the failure behaviour, a maintenance measure of the bridge superstructure is necessary after a service life period of 27 years. On the basis of the chosen assumptions and the related lifetime prediction, a significant reduction of the lifetime compared to the planned service life of the structure (80 years) is determined.

Tab. 4: Results of the reliability analysis carried out for the components of the bridge superstructure

Exposure	**Component**	**Limit state**	**Time until the limit state is reached (β = 1.7)**
chloride ingress (E_1)	carriageway slab (C_1)	critical chloride content at the reinforcement is reached	27 years
freeze-thaw attack (E_2)	webs (C_3)	2/3 of the concrete cover is destroyed	35 years
carbonation (E_3)	tension/compression chord (C_4)	carbonation front reaches the reinforcement	29 years

5.2 Combined deterioration mechanisms

In contrast to the calculation of the service life of each component of the bridge superstructure, the calculated system reliability is indicated in the following. The treatment of combined deterioration mechanisms can be realised by means of a fault tree analysis. Within a fault tree analysis the identification of the different failure modes of the structural components and their influences on the system can by performed. Hence, it is assumed that each component is either in a function state or in a failed state.

On this basis the state of the structure can be expressed in terms of the component functionality. The building structure usually consists of a large number of components which are connected in relation to their functions. The interaction of the different components of the structure influences the failure of the systems. The failure mode of one particular component may lead to the system failure. For instance, if the carriageway slab fails due to the corrosion of the tendons, the total super-structure of the bridge also fails.

The bridge superstructure is modelled as a series system. In terms of an undesired condition of the structure this system fails when component C_1 or component C_3 or component C_4 fails.

Fig. 4 shows the principle of this series system indicating components and relevant exposure conditions.

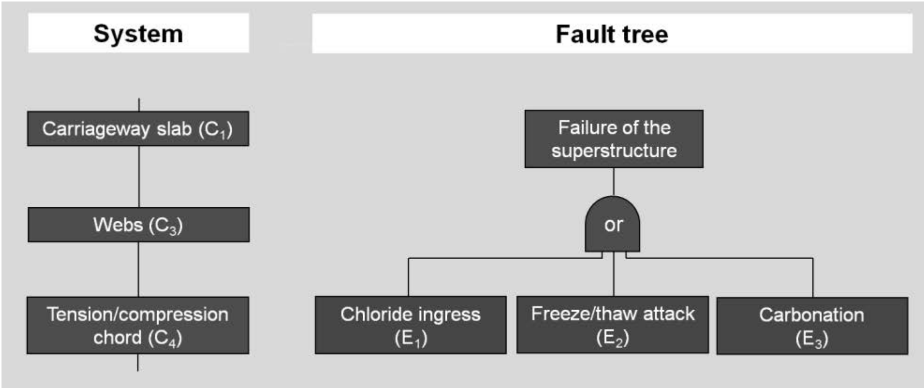

Fig. 4: Schematic illustration of the series system related to a bridge superstructure

The calculated result of the time-dependent system failure probability of the considered bridge superstructure related to the above mentioned boundary conditions using equation (4) is shown in Fig. 5. The upper curve (lower bound failure probability) is the result of the assumption that all failure events are statistically dependent. The lower curve (upper bound failure probability) is obtained when all failure events are statistically independent. Considering a series system it should be kept in mind that the system failure probability increases if the correlation between the failure events decreases.

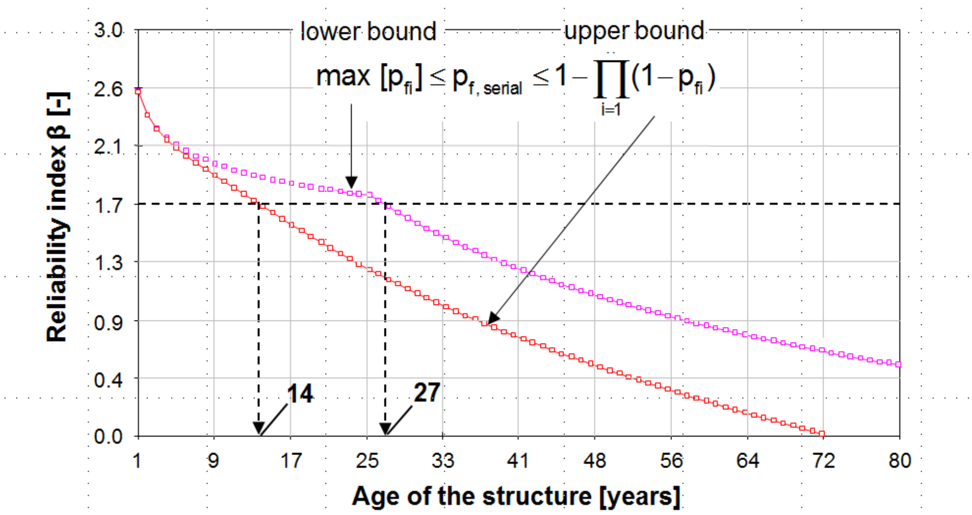

Fig. 5: Reliability index β vs. age of the structure on the basis of the system reliability investigation of a bridge superstructure

In comparison with the results of the service life prediction for the individual components the reliability analysis for the system of the superstructure results in a further reduction of the intended lifetime. Hence, not after a calculated lifetime of 27 years the structure has to be repaired but already after 14 years of service life maintenance measures are necessary.

6. INTERACTIONS

In the course of an extensive literature review the durability related exposures on concrete structures – concrete infrastructures like bridges, tunnels and hydraulic structures – were analyzed in detail (Müller, Vogel, Neumann, 2011; Schmid, 2013). In Tab. 5 the identified durability related exposures and their identified interactions are shown.

The exposures can either lead to the damage of the reinforcement in the concrete (e. g. carbonation induced corrosion) or to the damage of the concrete itself (e. g. freeze-thaw attack). For example, in the interaction matrix the interaction between the exposure of carbonation and chloride ingress is marked with the letter A_{carb}/A_{chlor} (see Tab. 5). Here, the process of the carbonation affects the process of chloride ingress and vice versa.

Further, one has to bear in mind that the chronological order of the occurrence of durability related exposures on concrete structures is relevant for the development of the degradation process. For example A_{chlor} means that the process of chloride ingress is the primary exposure and the carbonation of the concrete the secondary exposure.

Tab. 5: Interaction matrix for different exposures and attacks

Exposures	Carbonation	Chloride ingress	Freeze-thaw attack	Sulfate attack	Alkali-silica reaction	Singular risks
Carbonation	-	A_{carb}	B_{carb}	D_{carb}	G_{carb}	K_{carb}
Chloride ingress	A_{chlor}	-	C_{chlor}	E_{chlor}	H_{chlor}	L_{chlor}
Freeze-thaw attack	B_{frost}	C_{frost}	-	F_{frost}	I_{frost}	M_{frost}
Sulfate attack	D_{sulf}	E_{sulf}	F_{sulf}	-	J_{sulf}	N_{sulf}
Alkali-silica reaction	G_{alk}	H_{alk}	I_{alk}	J_{alk}	-	O_{alk}
Singular risks	K_{sr}	L_{sr}	M_{sr}	N_{sr}	O_{sr}	-

The above described combined occurrence of carbonation and chloride induced corrosion and their interaction were examined more closely in the following chapter.

7. MODELING OF INTERACTIONS

Fig. 6 shows the fault tree of a bridge element superstructure modelled for the deterioration mechanisms of carbonation and chloride induced corrosion, alkali-silica reaction (ASR) and for the singular risk of insufficient grouting of the tendon ducts which leads to corrosion of tendons. Here, the superstructure of the bridge represents a series system.

There are different possible effects which might be caused by the material-dependent interactions between carbonation and chloride ingress. On the one hand due to the carbonation the hardened cement paste might have an increased density and a lower porosity which can impede the further ingress of substances from the environment. On the other hand the binding capacity of the concrete is lowered due to the carbonation process. The total chloride concentration at the carbonation front might be higher than in non-carbonated concrete since the bound chlorides are released. In order to consider the material-dependent interactions between carbonation and chloride ingress the factor η_{carbo} is introduced, see Fig. 6.

7.1 Material-dependent interactions not considered

In the first step, the possible material-dependent interactions between carbonation and chloride ingress are not considered. Therefore, the so called interaction factor η_{carbo} is set to 1.0.

On the basis of the *fib* Model Code for Service Life Design (*fib* Bulletin 34, 2006) the limit state related reliability index β was calculated for different durability related degradation mechanisms. Appropriate values for the model parameters were selected from the literature (The European Union – Brite EuRam III, 1998; The European Union – Brite EuRam III, 2000; DARTS, 2004). For the deterioration caused by alkali-silica reaction and the corrosion of tendons corresponding failure probabilities were taken from an example of Zhu, 2008. The related value for an alkali-silica reaction (ASR) is $p_{f,ASR} = 0.5$ %, and for the corrosion of tendons the failure probability was assumed to be $p_{f,corr} = 2.0$ %.

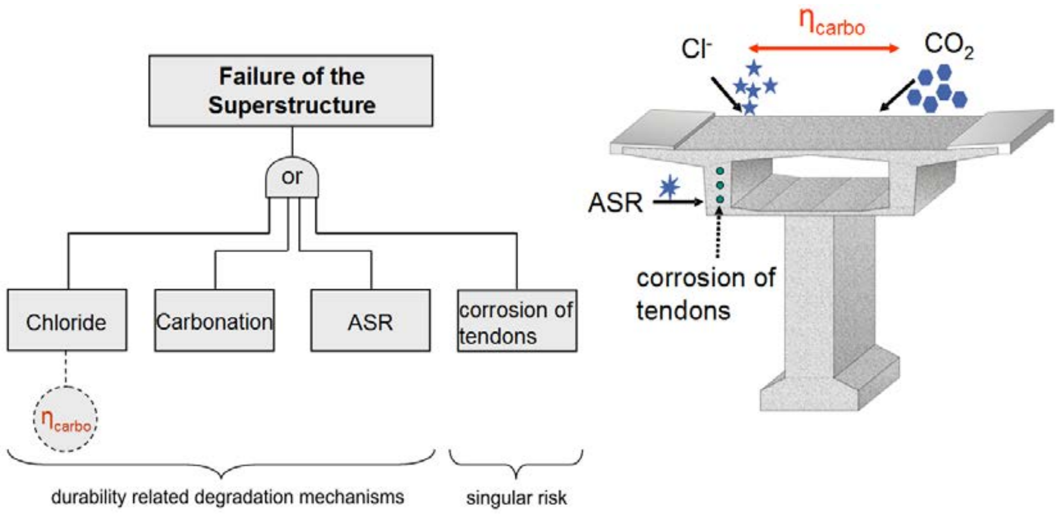

Fig. 6: Fault tree of the series system modelled for a bridge superstructure

The prediction of the system failure probability of the bridge superstructure was performed for a service life of 100 years. The target value of the reliability index is set to β = 1.6. Here, only the upper bounds of the series system were calculated, see Fig. 7.

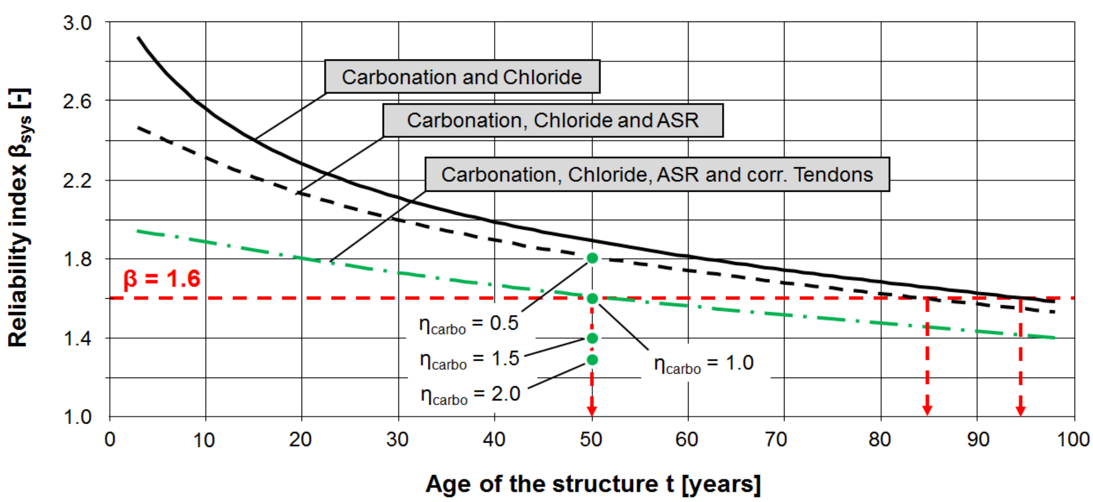

Fig. 7: Reliability index β_{sys} vs. age t of the superstructure of the bridge

The results displayed in Fig. 7 show that the reliability of the superstructure of the bridge decreases when the number of the deterioration mechanisms increases. For example the limit state (here $\beta = 1.6$) is reached after 95 years if only the deterioration mechanisms carbonation and chloride induced corrosion take place at the same time. In the case that all exposures are considered (carbonation, chloride, ASR and corrosion of tendons) the limit state is reached already after 50 years.

7.2 Material-dependent interactions considered

In the following example of calculation the interaction is limited to the influence of carbonation on the chloride ingress. For this study the beforehand introduced factor η_{carbo} was varied from 0.5 to 2.0. The parameter study was performed for a fixed service life of 50 years. The influence of an increasing (> 1.0) or a decreasing (< 1.0) factor η_{carbo} on the development of the reliability is shown in Tab. 6. If the average chloride diffusion coefficient is increased by the factor η_{carbo} being higher than 1.0, the reliability decreases, see also Fig. 7. Correspondingly the reliability increases with a decreasing chloride diffusion coefficient as a result of a factor η_{carbo} being lower than 1.0.

Tab. 6 Parameter study on reliability index βη and failure probability pf depending on the factor ηcarbo (fixed service life of 50 years)

η_{carbo}	Upper bound calculation	
	β_η [-]	p_f [%]
0.5	1.8	3.6
1.0	1.6	5.5
1.5	1.4	7.7
2.0	1.3	10.0

The results in Tab. 6 show that within the range of the varying factor η_{carbo} the reliability of the series system varies from $\beta = 1.3$ ($p_f = 10$ %) to 1.8 ($p_f = 3.6$ %). If the factor η_{carbo} is 0.5 (decelerated degradation process) the allowed safety level, expressed by the reliability index $\beta = 1.6$, is reached after 115 years. Otherwise if the factor η_{carbo} is 2.0 (accelerated degradation process) the allowed safety level is reached already after 15 years. It is clearly evident that the magnitude of the factor η_{carbo}, and therefore the extent of the interaction of chloride ingress and carbonation, has a very pronounced effect on the reliability of the series system of the superstructure.

7.3 Effect of cracks on chloride induced depassivation

The service life prediction of concrete structures based on full-probabilistic models concerning chloride induced depassivation of the reinforcement accounts for uncracked concrete (see above and (*fib* Bulletin 34, 2006)). However, cracks in concrete demonstrably influence the chloride ingress. This aspect can be verified by both laboratory and field investigations. Field investigations on cantilevered parapets on a concrete bridge revealed an increase in chloride diffusion in the crack zone with crack widths of 200 μm and 300 μm by a factor of 2.6 and 2.8, respectively, compared to uncracked concrete. Consequently, the implementation of a crack factor f_{cr}, which has been quantified by field investigations as a function of crack width, has to be taken into consideration (Schmiedel, Vogel, Kotan, Müller, 2018). Service life assessments based on a correspondingly extended approach resulted in a significant increase of failure probability for the crack zone compared to uncracked concrete. Further work on this subject is under way.

8. CONCLUSIONS

Currently, the procedure for durability design of concrete structures which is given in national and international standards is only a descriptive approach. This approach includes considerations of prescriptive limits relating to the concrete composition and concrete cover as well as the curing. In contrast to the descriptive approach the advantage of the performance approach for service life design is based on the fact that the durability of concrete structures can be quantified in terms of failure probabilities corresponding to a defined reference period.

The procedure for durability design, i.e. probabilistic service life design as given in the *fib* Model Code for Service Life Design (*fib* Bulletin 34, 2006) is focused on the essential deterioration mechanisms of concrete structures, which are treated individually. The knowledge and the design methods established so far have to be extended to consider interacting instead of single actions and therefore to deterioration models which incorporate related effects. A first approach with regard to the modelling of interactions has been developed and presented here. It could be shown that the reliability of a system depends not only on the reliability of the individual components but also on their interrelation. This fact has to be taken into account when the service life of complex structural systems should be estimated.

Although the practicability of the new approach for durability design has been verified by a sophisticated computer based example, it is obvious, that there is still a high need for extended research. To understand the influence of combined environmental impacts on durability of concrete structures wide numerical analysis has to be performed with modified deterioration models. In addition to the probabilistic analyses the aspects of the material properties relating to the interactions have to be examined also in the course of laboratory and field investigations.

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EUROPEAN TRENDS IN PAVEMENT MANAGEMENT – A CHALLENGE FOR AUSTRIAN ROAD ADMINISTRATIONS

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SUMMARY

In 1998 the implementation of the Austrian PMS started as a big challenge for the Austrian Road Administrations. Since 1998 the PMS will be used for analyzing the motorway and expressway network and also on a high percentage of state roads, community roads and rural roads. The original expectations from 1998 to use the PMS as decision support tool for the definition of a short- to long-term pavement maintenance program are still existing, but new requirements for a modern asset management solution in a digital word are extending these tasks more and more. Thus, the following paper presents the challenges of using a holistic pavement management approach in the new world of digitalization and innovative technologies.

1. INTRODUCTION

In 1998 the implementation of the Austrian Pavement Management Systems (short PMS) started as a big challenge for the Austrian Motorway Company ASFINAG, the Federal Ministry of Transportation, Innovation and Technology in cooperation with the nine Austrian states. Since 1998 the PMS will be used for analyzing the motorway and expressway network (ASFINAG-network, including the federal roads category A and S with a network length of approx. 2.200km) but is also applied in practice on a high percentage of the state road network with a total length of approx. 28.000 km. Community roads and rural roads are the most recent networks for the PMS analysis. The following Fig. 1 gives an overview of the PMS application in Austria on the different types of road networks

Motorways and expressways (Federal Roads)

ASFINAG	2200km
Bonaventura	50km
State roads (category B and L)	
	800km
	1700km
	1400km
	6000km
	13000km
	550km
	4900km
	28350km
Communities and rural roads	10600km

Fig. 1: Overview of PMS application in Austria

The PMS from 1998 was implemented as a stand-alone solution with a comprehensive pavement database, including all necessary information and data for running life-cycle-cost analysis (LCCA) on the technical level. For the practical implementation the Austrian Road Administrations decided to use the commercial Canadian software solution dTIMS (Deighton Total Infrastructure Management System), which is still in use for the analysis of the networks as listed in Fig. 1.

Pavement Management and Pavement Management Systems (short PMS) became a standard approach in the asset management process of most of the Austrian Road Administrations. Especially, on high level road infrastructure networks, like motorways and expressways, the use of the PMS for the selection of maintenance treatments on section or object level but also for the assessment of strategic targets and requirements on network level is state of the art.

Nevertheless, the use of a holistic pavement management approach, is a challenge for road administrations and the involved decision makers. The main objective is also on this level the provision of a pragmatic and repeatable solution, which gives the road administration as well as the decision and policy makers the possibility to underline necessary investments into their road infrastructure networks.

2. ACTUAL REQUIREMENTS FOR A MODERN PMS

2.1. The asset management process

The asset management process, which is schematically shown in the following Fig. 2, gives an overview of a holistic asset management framework. On the different types of roads, the expectations of the different stakeholders (users, neighbours, road owners and operators, environment, etc.) define the requirements and specifications, which should be an integrative part of the maintenance policy and the strategic targets finally. To combine the strategic level with the technical requirements it is necessary to support the road administration from the communication and the organization point of view. Thus, the Austrian PMS can be defined as a decision support tool, which enables to answer the different "maintenance questions" on the different levels (Weninger-Vycudil, 2017).

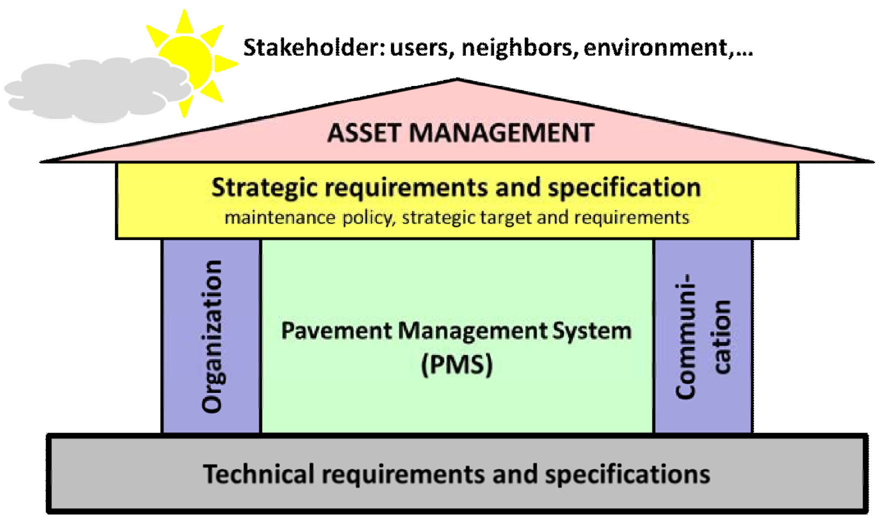

Fig 2: Schematic asset management framework

To guarantee a successful approach of the PMS, the pavement management process must be integrated into a holistic asset management solution, considering the following activities and tasks:

- Management of data and information (monitoring and collection, interoperability, data storage, data exchange, etc.)
- Analysis of data and information using future oriented analysis methods (life-cycle-assessment, life-cycle-cost-analysis, life-cycle-risk-analysis, etc.)
- Reporting of data and analysis results (reports, graphs, maps, web-sites, etc.)

An overview of the different tasks of a pavement management process can be taken from the following Fig 3.

Fig. 3: Pavement management tasks (schematically, graph Deighton Associates Ltd.)

2.2. Objectives and benefit of a PMS

For a successful PMS implementation, the effort needs to be compared with the objectives and finally the benefit of the practical solution. Based on long-term experiences of PMS application in Austria, the following list gives an overview of objectives, which can be achieved (Weninger-Vycudil, 2017):

- Increase of efficiency based on a systematic and objective planning of maintenance treatments.
- The basis for a sustainable solution is the knowledge about the actual pavement construction, including different types of data, which are describing the pavement characteristics from the maintenance point of view.
- Integration and assessment of strategic targets into the maintenance process

- The PMS is still a technical approach, which should answer the following questions:
    - Which maintenance treatments on which road sections?
    - When is the best point of time for the maintenance treatments?
    - Where should it be done?
- Evaluation and estimation of asset value
- Provision of a basis for the realization of short- to long-term maintenance (investment) programs

By providing the results of a PMS, the benefit can be seen on different decision levels. On the technical level, the pavement management engineers should be able to use the objective recommendations as a basis for the definition of the maintenance treatments on the different road sections. On the strategic or management level, the network level results enable to assess the achievement of strategic targets and requirements and to predict the future maintenance needs of the assessed networks. Especially the discussion of necessary investments can be carried out on an objective base, showing the consequences and the effects to the different stakeholders.

3. THE CHALLENGES

3.1. The digital world – the 4th Industrial Revolution

3.1.1. What is digitalization?

The development of digital technologies is changing the world at a rate and on a scale not seen before. Social scientists define this progress as the 4th industrial revolution and are using the term "Industry 4.0" to describe these trends. Industry 4.0 labels the current developments of automation and data exchange in technologies. Global data networks like the world-wide-web, are disrupting the way decision are being made, with both benefits and risks.

The trend of digitalization can also be seen in asset management. On the one hand in the design or maintenance of new or existing assets and on the other hand in the day to day tasks of road operation. The basic principle of digitalization is a process, where information will be converted into a digital format, which can be used by computers. In the context of asset management, digitalization converts properties and characteristics of the infrastructure assets (inventory, condition, etc.) into digital data and enables the management of this data by using asset management software tools. Digitalization is related to different tasks linked with processes, especially in the context of data quality, management and analysis. The benefit can be manifold but requires an efficient use of the digital information from the beginning (Weninger-Vycudil, 2019).

The basis for digitalization is data. In the last decades many data collection activities have taken place, starting with pavements, bridges and tunnels. Inventory data were taken from drawings and plans, and often stored in complex asset specific database systems. The first asset management systems started as individual solutions (PMS, BMS, etc.) taking only this information into account, which could be directly addressed to the asset category and the respective tasks. Analysis solutions were related to analyze single assets using ranking or prioritization models to define the necessary maintenance treatments. Only a few applications offered the possibility to link data from different assets or from different phases of the whole life-cycle process. The conversion of information into data is still an ongoing task, especially for those asset categories, which are showing less importance in the maintenance processes

(Weninger-Vycudil, Piane, 2019). Thus, the collection of (new) data is still a challenge in moving forward for road administrations, although most of this data are stored somewhere in the digital world.

3.1.2. The road into digitalization

The basis to successful digitalization involves a clear understanding of the different life-cycle phases of infrastructure assets, considering the data needs and requirements:

- The *planning and design phase* is the starting point of digitalization. Digital planning is the state of the technology almost everywhere, where the basic information about the inventory, the construction, the materials, etc. will be produced.
- The *construction phase* uses data from the planning and design phase and extends it with actual information from the construction.
- The *maintenance and operation phase* covers most of the service-life of an asset. Changes on inventory and condition must be collected and integrated into the process. A holistic asset management framework covers both, the operational and the maintenance tasks. It includes all kind of activities, from routine maintenance to heavy rehabilitation, but also organizational activities like planning of inspections, data implementation, measuring of performance, etc.
- The *reconstruction and recycling* phase characterizes the end of the service-life of an asset. The knowledge about actual materials becomes a decisive factor from the recycling and the new planning point of view.

A successful way into digitalization requires the use of digital information in all phases. At the moment, the maintenance phase is often a stand-alone solution, starting with the basic data collection as the initial task. Of course, many assets have never been digitally planned or designed and existing (old) data formats do not fulfil the requirements of data storage in the maintenance phase. These problems must be solved in any case and will be one of the major challenges for the new PMS.

3.1.3. The benefit of digitalization

Digitalization is a starting point and not the solution. The planning of maintenance activities, the estimation of the maintenance needs, the reduction of maintenance backlogs, etc. are still the objectives of asset management. Digitalization is an instrument, providing an objective and understandable base for the different levels of users (engineers, managers, customers, etc.). The benefit is strongly related to an efficient use of data. Digitalization is more than data storage, it includes the use of data in the assessment and analysis processes to the maximum possible extent. Thus, the benefit of digitalization can be summarized as follows (Weninger-Vycudil, 2019):

- Provision of digital information for managing the assets
- Support of decision processes on different levels (project, network, strategic)
- Improvement of internal and external communication
- Assessment and analysis of real-time situations, with extrapolation (prediction) of future situations for different what-if-scenarios
- Combination of data from different sources (interoperability)
- Controlling, adjustment and improvement of infrastructure maintenance and operation processes
- Understanding of relationships and mapping of life-cycle phases
- etc.

3.1.4. The Risk of Digitalization

Digitalization includes complex processes with inherent risks that need to be managed from the beginning. Especially, the quality and quantity of data is often the critical factor for a successful implementation. The main risks can be categorized as follows (Weninger-Vycudil, 2019; Weninger-Vycudil, Piane, 2019):

- Data quality
- Data quantity (from Big Data to Smart Data)
- Management of incorrect and incomplete data
- Data sources using different formats and/or referencing systems
- Data accessibility and security
- Performance of asset management software solutions
- Data communication, visualization, user interfaces

It is necessary to analyze the risks before starting an implementation process. In many cases the organizational structure of the administrations offers an additional risk, which is independent from the technologies to be applied but can be a decisive factor for the use of the new technologies in the context of digitalization.

3.2. Fields of application for a PMS

Strategic and technical pavement management can be applied in different fields. Nevertheless, it is important to define the main fields of application and to concentrate on problems, which are in close connection with these areas. There is a huge number of challenges in the whole asset management process, where a PMS can support the different users and stakeholders. Based on the experience of actual research activities (CEDR Conference of European Directors of Roads, TRB, etc.) and cognizable trends, the following main fields of application can be drafted based on (Weninger-Vycudil, Piane, 2019):

- *Safety and security*
  Data and analysis results are powerful instruments in increasing the safety but also the security of the pavements. It is a strategic goal in every infrastructure policy to improve the safety and the security, given the current situation of potential threats.
- *Reliability of infrastructure*
  The user of the infrastructure, which is one of the most important stakeholders in this process, expects a high availability of the infrastructure. Thus, pavement management needs to address these requirements as much as possible. The public as a new user should have access to asset management systems. A permanent (data-limited) communication between the database of an asset management systems and the user (e.g. navigation system, vehicle computer) sounds futuristic, but will be an important task for a holistic framework.
- *Economy and effects of maintenance activities*
  The maintenance budget is still the decisive factor in the decision process. A pavement management system gives different options (alternatives) and shows the effects of maintenance strategies in monetary and non-monetary terms. The question "What is the most efficient solution?" can be assessed from different point of views, where different analysis models in a PMS should be offered (economy based, risk based, multi-criteria based, etc.).

- *Resilience and sustainability*
  With regard to the results of the 2015 United Nations Climate Change Conference, COP 21 (Paris), the effects of a changing environment have become, more and more, a serious problem of the whole world. It is a fact, that the effect of climate change will have a significant impact on the design and management of our (transportation) infrastructure. No longer can we assume the environment to be a static variable in our analysis. Like condition, the environment must be modelled and treated as a dynamic analysis variable. Thus, it will be necessary to apply maintenance methods and strategies, which allow to assess the energy balance and the degree of sustainability and to include these types of analysis into the configuration of the asset management systems. Only a modern PMS, which gives answers on this kind of questions, will have a high acceptance in the whole process. Beside economic aspects, future decisions will be based on the environmental efficiency and the PMS must offer the possibility to integrate these models in a proper way (e.g. energy labelling).

3.3. Levels of Application

It is a decisive question about the different levels of application within a holistic pavement management framework. A certain answer can be derived from the term “life-cycle-assessment”, which includes the world “life” and which can be transferred to the whole life of the pavement. An important task is the starting point. In many PMS, the starting point is the starting of the infrastructure operation. Nevertheless, it is not the starting point of the life of an infrastructure asset. Thus, the levels of application in a PMS starts much earlier, because important information for the whole life-span of an asset will be generated by drawing the first line. In addition, the ending point of an asset is not the demolition. The pavement materials from the demolition should be recycled to the highest possible extent and thus, needs to be addressed in the PMS as well. The main levels of application within a holistic asset management framework can be summarized as follows:

- *Planning and designing the assets*
  Design is the beginning of the process. Selective data from this phase (e.g. BIM-data) needs to be stored in the PMS data base and should be used for the life-cycle-assessment of different design scenarios, which gives the designer, as the operator will eventually do, the possibility to look into the future.
- *Constructing the assets*
  It is important to collect the first actual information of an asset, when it is under construction. In this phase, access to the PMS enables the documentation of the construction and to get the first inventory data in a very effective way. This could be covered by using BIM-data, which will be used in the construction phase as well. Especially, material information (e.g. from performance based design) could be directly used in the performance prediction models for calibration.
- *Maintaining and operating the assets*
  Infrastructure operations and classical pavement management need to be combined comprehensively. A holistic asset management framework covers both, the operational tasks and the maintenance tasks. To be effective, it must include all kind of activities, from routine maintenance to heavy rehabilitation, but also organizational activities like; the planning of inspections, data implementation, measurement of performance, etc., of course fully supporting the strategic targets of the infrastructure agency.

- *Reconstruction and recycling of the assets*
  Regarding the collected information from the previous three phases the reconstruction and recycling of the assets should be designed, planned and scheduled as a part of the asset management process. It closes the loop of the life-cycle also from the asset management point of view.

3.4. Data changing the world

3.4.1 Managing new data and information

The addressing of data and information is still one of the most important tasks in the pavement management process. Within the last decades the way in which a user communicates with data has changed. Especially in the area or data visualization and mapping. These areas gained significant importance in comparison to traditional tables and lists of data. Also, the number of different data sources is growing and terms like big data, smart data, fast data, etc. are buzz words in the world of data.

The combination and communication of data from different sources will define new requirements for modern asset management. Databases will become communication platforms between different sources, where the origin data should stay at the origin place. Thus, a pavement management database must be able to communicate with all kind of basic information, like spatial information from BIM or from network information systems (e.g. Web information used in the PMS, see Fig. 4).

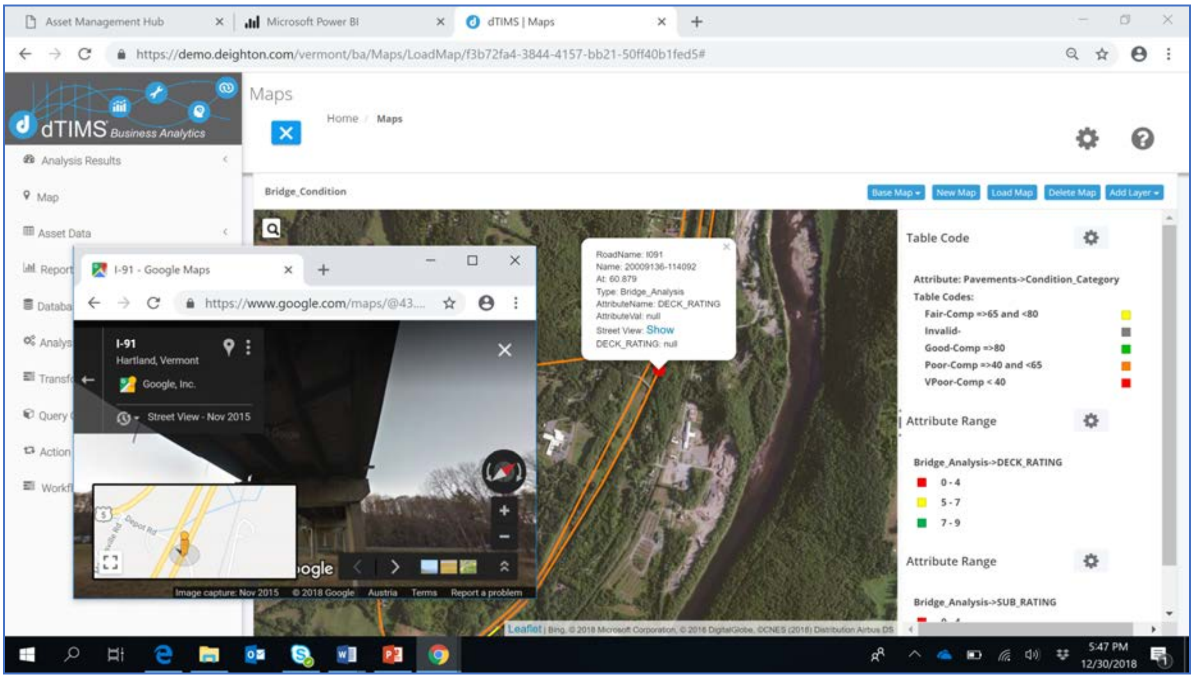

Fig. 4: Integration of Google Street View into the PMS

Building information modelling (BIM) has become a standard in the design and construction processes of infrastructure assets. In many countries public tenders for road design projects test how BIM will work successfully in this field and actual research projects look at how BIM can work together with asset management. Thus, a modern pavement management solution should be able to edit, view and adjust this data and to link it with existing information from the assessment and analysis (condition, maintenance treatments, etc.).

3.4.2. Selection of Data

The amount of data is still growing, and modern pavement management systems should avoid producing data graveyards, where data will be stored only. For example; every minute, 48 hours of video are uploaded onto YouTube, 204 million e-mail messages are sent, and 600 new websites generated. The selection of data is a challenge for all parties to be incorporated:

- *Useful Data*
  The selection of useful data requires a high flexibility in the design of databases. New types of tables, attributes and relations must be checked about their added values and should be implemented if necessary. The objectives of using specific data must be defined from the beginning, considering future needs and demands.
- *Big Data*
  The amount of data from permanent data collection processes will increase progressively during the next years. New methods of data collection in the different fields are under development or testing. E.g. the use of drones for condition inspection is a cheap and easy technology and will bring up more and more data in short term.
- *Smart Data*
  Smart Data means information that actually makes sense. It is the difference between seeing a long list of numbers referring to machine data of a measuring vehicle/drone or the condition data being summarized in the form of 100m section data. The transformation of Big Data into Smart Data will be a task of both the devices, which collect the data but also for the PMS. This is the only way to analyze assets from smart decision process point of view.
- *Fast Data*
  As-it-happens information enables real-time decision-making. This type of data needs to be linked with automated or semi-automated decision processes. Work-flow based data management using self-paced learning procedures (as a part of AI) will be necessary to handle Fast Data in an efficient way.

3.4.3. Interoperability

Interoperability is the key word in digitalization and thus a key element into the smart data management process. Although many road administrations are still setting up new databases and collecting information from scratch, the definition of links and data interfaces is a challenge for the successful entry into the digital world.

There is no single world-wide database, which stores all the information of the internet. Asset manager must understand this principle, otherwise digitalization becomes extremely cost intensive and inefficient. Of course, the assessment of data quality is a critical task and requires an understanding of the information to be used in the different processes. A focus in a PMS is to link different data sources and to find a common language for the exchange of information. The following Fig. 5 shows the combination of pavement condition data with accident information from an internet source on a map.

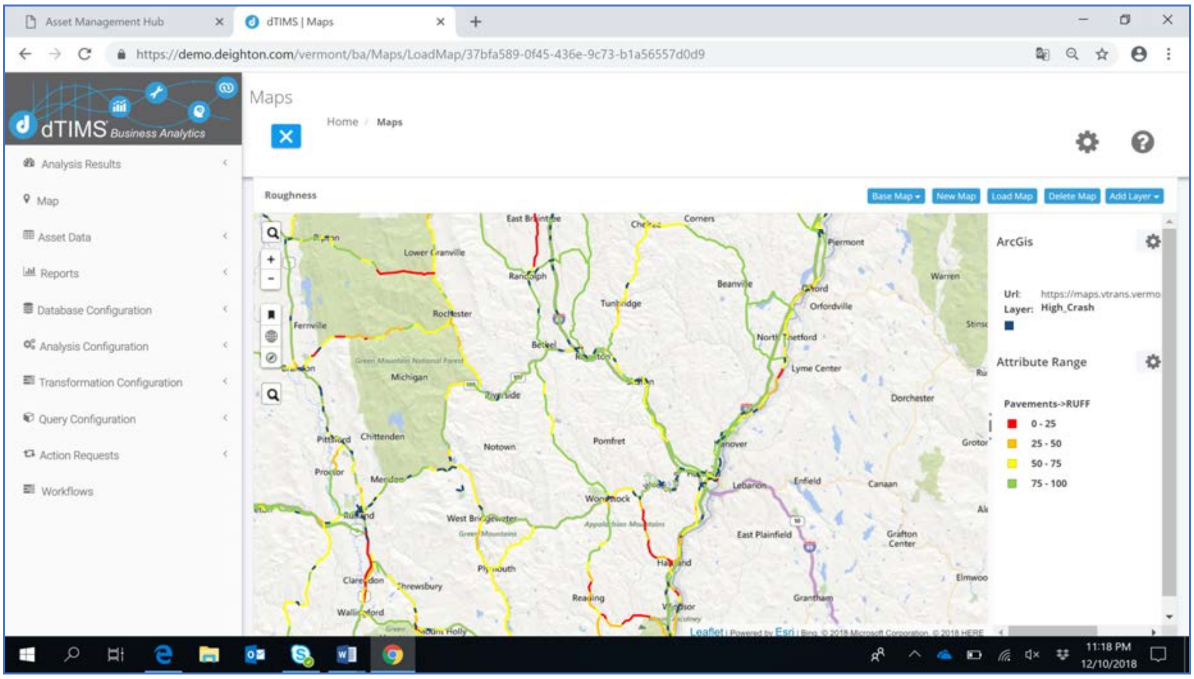

Fig. 5: Combination of pavement condition data stored in the PMS with external accident information (web-link) on a map

3.4.4. Smart Data Management

The management of data from different sources so that an effective pavement management and a smart decision process is possible, is becoming a large challenge. Bring the data together means, beside an advanced communication, that there must be a clear and repeatable referencing of the data in the PMS. Different data sources can be referenced in different ways, where the classical linear-referencing and geo-referencing needs to be merged and extended to other referencing methods like 3D information coming from BIM or drone-based data collection devices.

A decisive factor in how we become smart at managing our data is the storage of information or how information from different sourced can be linked. Traditional relational databases will be replaced by semantic solutions, where semantic data management enriches and links heterogeneous data. Graph databases use graph structures for semantic queries with nodes, edges and properties to represent and store data. A key concept of the system is the graph (or edge or relationship), which directly relates data items in the store a collection of nodes of data and edges representing the relationships between the nodes. The relationships allow data in the store to be linked together directly, and in many cases retrieved with one operation. Graph databases hold the relationships between data as a priority. Querying relationships within a graph database is fast because they are perpetually stored within the database itself. Relationships can be intuitively visualized using graph databases, making it useful for heavily inter-connected data.

Intelligent filtering from different sources (enterprise data, social media data, sensors, etc.) will be a key component of the smart data management concept and thus an integrated part of a PMS. It allows us to link the PMS with the world of data avoiding the use of so called “unstructured” data.

3.5. Smart decision making

3.5.1 How to make smart decisions?

A detailed knowledge about the effects and consequences caused by decisions (on different levels) is one of the major goals in the business world. In many cases the knowledge is not available, limited or cannot be retrieved or deviated from available information, although this information can be found in the big world of data.

Smart decision making using new and advanced technologies (e.g. AI) will be able to find, filter and provide the necessary basic information in an understandable way (e.g. in form of understandable indicators) and integrate the necessary (smart) information into the decision process. This approach is based on a continuous learning process, where a part of the learning is up to the system and their tools using all available smart data. The user profits in medium- and long-term from this approach, where the results of previous solutions will be validated by assessing their effects and consequences (from different point of views, see KPIs). In addition, the decision space (area of decision) can be reviewed for alternatives, where effects and consequences are already known. In principle, answers can be given without asking a (new) question. The concept and finally the PMS will help the user to look in new directions (methods, technologies, etc.). As a summary, the smart decision-making concept offers the following possibilities:

- Select that information, which you really need in the decision making process from the world of data
- Assess solutions by getting detailed information about the effects and consequences based on the validation of previous decisions (and their effects and consequences shown in the data sources).
- Provide alternative solutions by getting detailed information about the effects and consequences based on the validation of previous decisions using this alternative approach (and their effects and consequences shown in the data sources)
- Benchmark the basic solution with the alternatives

As a starting point, the system must learn how to make decisions, where the user plays the role as the teacher at the beginning. If the system has learned the lessons, it should support more and more the decision makers in finding better solutions. Complex and time-consuming simulation can be avoided but are not excluded (can help to support especially at the beginning). This is the basic principle of artificial intelligences (AI), which is one of the technologies to be used in PMS in the future. To which extend such process can be automatized is strongly dependent on the responsibilities, the quality of the underlying information and if the user trusts the system more than his own opinion (which leads to a non-rational discussion).

3.5.2 The elements of the smart decision-making process

The key understanding of a smart decision concept is based on the process of how decision can be made in the digital world, considering the benefit of having access to a progressively growing data source, so called the web. As already mentioned, there is a need to use smart data management processes to get efficient access to the necessary information using new and innovative technologies. The following elements of a smart decision-making process can be identified and will be an integrative part of PMS in time (Weninger-Vycudil, Piane, 2019):

- *Smart data management*
  As already mentioned, intelligent filtering from different sources (enterprise data, social media data, sensors, etc.) will be a key component of the smart data management concept.
- *Decision support*
  An intelligent analysis of smart data is the basis for future decisions on the different decision levels (strategic, tactical, operational), which are linked with each other to the necessary extend. The classical analysis becomes more and more automated and learns from analyses, which have been done before. New or extended indicators should be used on different levels and must address the knowledge of the different decision makers (e.g. asset value for financial manager, environment for the environmentalists, risk for risk manager, technical indicators for engineers). Life-cycle planning must be connected with risk analysis to life-cycle risk analysis (LCRA) and multi criteria analysis (e.g. balance effect analysis BEA) will become state of the art.
- *Decision validation*
  To close the loop and learn from decision from the past a comprehensive decision validation will be necessary and should be an integrative part of the PMS. Also, in this field intelligent validation of decisions and their effects in the different areas and on the different decision levels are an extended input for new decisions and the selection of data in the smart data management process.

4. NEW TECHNOLOGIES ARE THE DRIVERS

The management, assessment and analysis of a high number of different data is a critical factor in digitalization and the world of smart decision management. Only new technologies can support the users and operators on the different levels and enable them to handle the data, the analysis and the validation of the decision in an efficient way in the PMS.

A forecast on new technologies is difficult because of the complexity of the technology world and the high number of successful and less-successful ideas. In this context, 3 main directions have been identified, where the author sees the highest influence on PMS in the next few years. The selected fields are explained as follows.

4.1. Artificial Intelligence (AI)

Artificial Intelligence (AI) can support the users on different levels, considering experiences from previous processes. The "machine" will be able to support processes like data selection, data preparation, data analysis and decision validation based on lessons learned before (from instructors or previous processes and decisions). Self-paced learning can significantly reduce the effort in managing data, where the quality of the result is the main indicator for the quality of the approach.

4.2. Augmented Reality (AR)

The combination of digital information with the real world is the main objective of Augmented Reality (AR). Since "Pokémon Go" AR is state of the art. In the same way data from PMS can be forwarded on site. The road operator sees on his smart phone application (mobile app) the work order for the pavement, executes the request and closes the order. The system guides the operator to the right place, through the right process and finally collects the effort to fix the issue. Furthermore, AR enables the user to view information on-site, by a digital projection of data on the real asset (finding damages, distresses, etc.).

4.3. Smart Visualization

The next generation of visualization allows the user to communicate interactively with data and analysis results. The user will be able to view relations and search for data on different levels of detail using asset management data or other sources (e.g. BIM). Modern Business Intelligence (BI) systems and dashboards support this form of data visualization, using different types of graphs and dialogues (example, see Fig. 6).

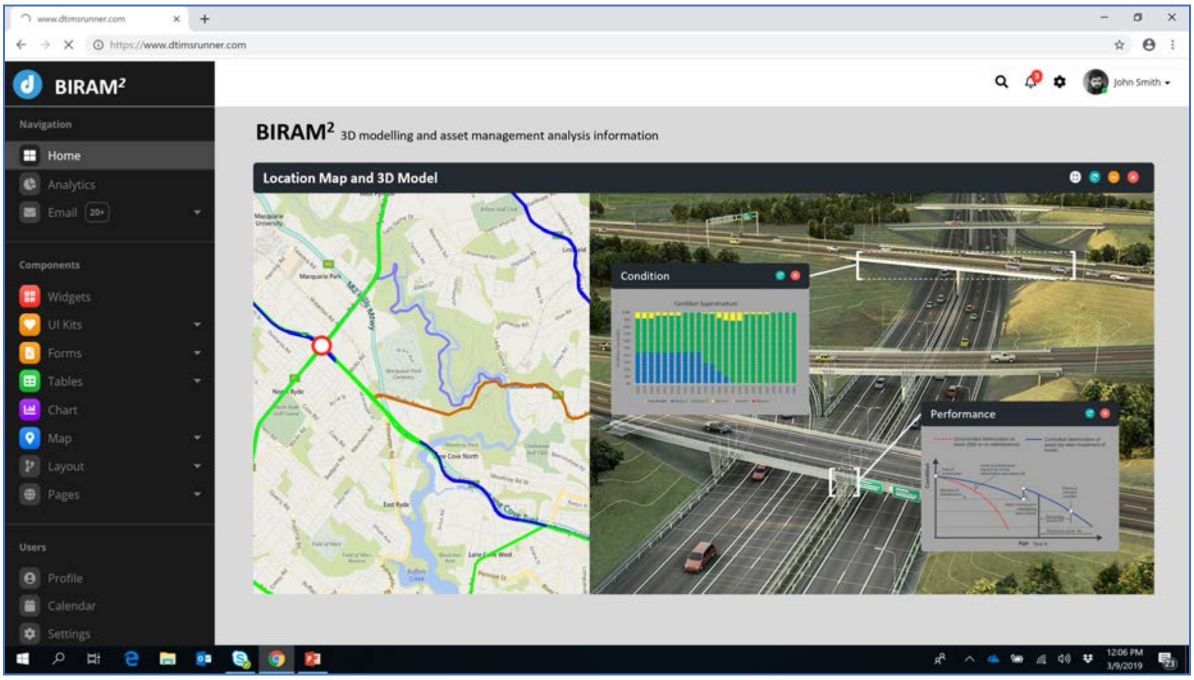

Fig. 6: New way of visualization (BIM data in combination with AMS analysis results – design study)

5. CONCLUSION

Digitalization in pavement management is the most challenging trend for the future. The use of an objective digital basis for a sustainable and future oriented maintenance planning underlines the motivation of moving into the digital world. Especially the integration of new technologies (e.g. AI, BIM, AR) will enable to manage the increasing number of data in the different processes. Existing PMS must do this step based on the lessons learned from the past and from successful PMS implementation like in Austria.

At the moment, the Austrian Road Administrations review the different possibilities of moving to a holistic asset management solution, which brings the maintenance needs of all kinds of assets together. A PMS becomes an AMS or is an integrative part of a comprehensive solution. The digital world will make pavement management more complex, but a good PMS will be still able to manage the information and provide the data in an understandable way, so that the decisions can be based on an objective and more effective point of view.

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THEORETICAL AND EXPERIMENTAL ANALYSES OF HISTORICAL DANUBE BRIDGES IN BUDAPEST

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SUMMARY

This paper gives an overview on the rehabilitation of historical Danube bridges, which are located in the UNESCO World Heritage part of Budapest: Széchenyi chain bridge, Margaret bridge and Liberty bridge. The bridges are briefly introduced and the special aspects of the design of reconstruction are shown. Site measurements and advanced studies, which supported the rehabilitation in the design and construction phase, are discussed in the case of the two recently reconstructed bridges and for one planned case. The paper also introduces specialties of the reconstruction of old historical bridges and presents advanced solution methods for their static investigation based on numerical simulations and on-site tests.

1. INTRODUCTION

In the history of Budapest the bridges act major role. After 25 year the opening of the first permanent bridge on the Danube, Budapest was formally created by the union of the two cities, Pest and Buda, on the East and West banks of the river in 1873. Since then the extension and the development of the city is essentially influenced by the new bridges (Gál, 2005; Wikipedia, 2019). Today in the Budapest region altogether 11 bridges serve the continuously increasing traffic. One-one highway and railway bridges are located on the North and South edges of the city and seven road bridges are in the city. Four of them are in the downtown area and these are the oldest road bridges of Budapest: Széchenyi chain bridge, Margaret bridge, Liberty bridge and Elizabeth bridge, which were built in the 19th century (1839 – 1903). The first two bridges were designed by foreign designers, and the other two by Hungarian engineers. All of them hold the natural beauty of the applied structural system with aesthetic appearance in the very heart of the city. All these four bridges belong to the UNESCO World Heritage Castle Hill and Danube bank part of Budapest. All the four bridges serve the transportation of the city for more than hundred year. Among the four downtown bridges three were rebuilt after the war by the same structural system, the Elizabeth bridge, however, got a new structure. Since these structures not just serving the traffic but also parts of the beauty of the Budapest, there is a predestined focus on these aging bridges. In the mid 2000's a global rehabilitation program is started on them: the Liberty and the Margaret bridges were reconstructed in the last decade, and the rehabilitation of the Széchenyi chain bridge will start next year. This paper has a focus on these three historical bridges with special aspects of the design and the related site measurements, load testing and advanced structural analysis.

2. SZÉCHENYI CHAIN BRIDGE

The Széchenyi chain bridge, built between 1839-49 was one of the most attractive monument of his age (Domanovszky, 2015). With its middle span of 202.6 m it was the largest chain bridge of its erection time, and still it is the third one after the Herzilio Luz bridge (339 m) in Brazil and Clifton bridge (214 m) in the United Kingdom. The original structure of the chain bridge is shown in Fig. 1. The designer of the bridge was the famous engineer and bridge designer of that age, William Tierney Clark. The original steel chains were manufactured in England, which had a total of 2000 tons' self-weight. The bridge was manufactured with steel cross girders but without stringers (stiffening girders) at the bridge deck. The original deck system was made of wood. Thus significant vibrations are observed on the bridge, the complete strengthening was decided and executed between 1914-1915 based on the graphostatic analysis of Prof. Antal Kherndl. During the reconstruction the chain system was replaced, the number and size of the chain elements are increased and the bridge deck system was extended by truss stiffening girders. The bridge was destroyed in the Second World War, and completely rebuild between 1947-1949 in the same form and structural system, as shown in Fig. 2.

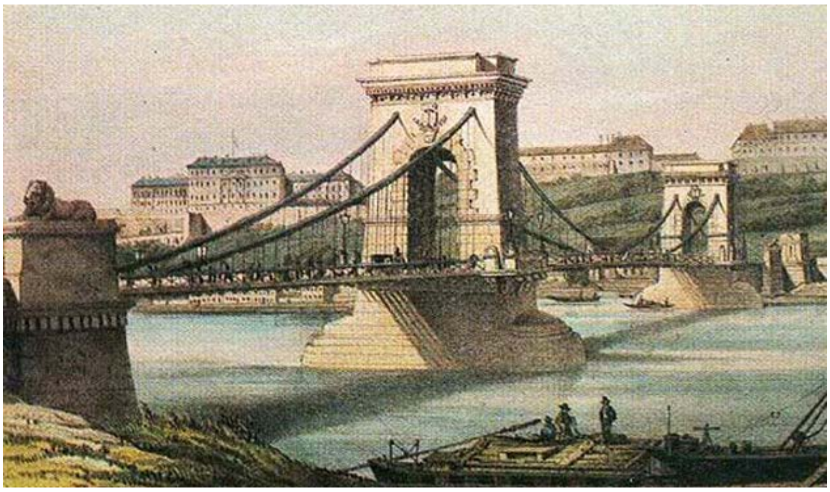

Fig. 1: The original Széchenyi chain bridge

Fig. 2: The reconstructed Széchenyi chain bridge

The reconstruction of the structure become necessary nowadays again, because significant corrosions are identified in the stringers of the bridge deck under the reinforced concrete deck. The lost in the cross-sectional area reached 40% at certain locations and the condition of the reinforced concrete slab become also critical. Therefore, it was decided to replace the reinforced concrete deck with a steel orthotropic deck system, as shown in Fig. 3. The chain system, the stiffening girders and the cross girders were not in critical stage, therefore they can be kept under the reconstruction. The general concept and high priority during the reconstruction is to keep the historical attitude of the bridge. The design stage is executed between 2015-2016 and the reconstruction will start in 2020 (Nagy et al., 2015).

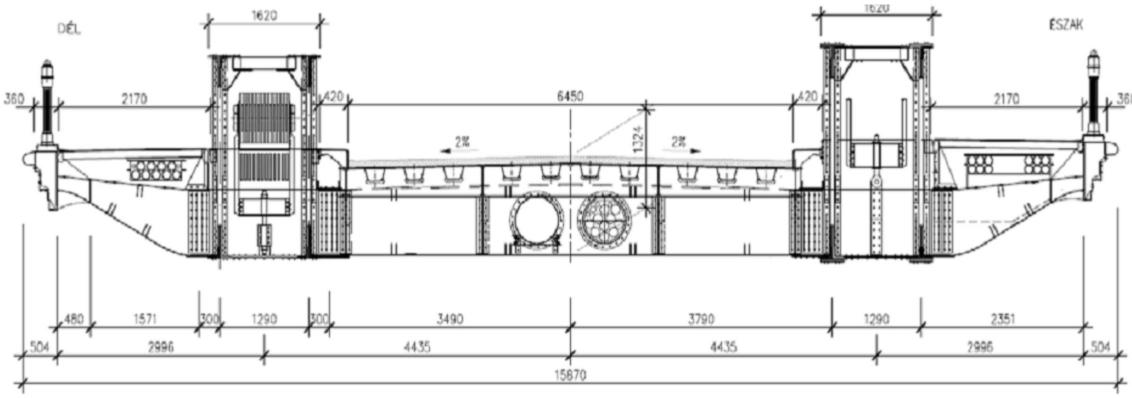

Fig. 3: Cross-section after the reconstruction

During the project numerous specific structural problems and modelling specialties turned out, which comes from the layout of the old historical structure, long time traffic and corrosion, which required a detailed structural analysis and on-site experimental testing. Corrosion is a significant problem for old steel bridges, therefore corrosion measurements are executed on the Széchenyi chain bridge by the Department of Structural Engineering, Budapest University of Technology and Economics in 2002. The literature survey on historical chain bridges called the attention on the corrosion fatigue of the chain elements. The Silver bridge built in 1928 over the Ohio river in the USA was collapsed in 1967; the reason for that was that fatigue crack occurred in the chain elements due to corrosion between the pins and the chain. The corrosion resulted in fixed connection instead of the originally pinned joint for the chain having large stress concentration at the location of the pins (Åkesson, 2008; Cullimore et al., 1993). A detailed investigation is executed in case of the current reconstruction of the Széchenyi chain bridge containing corrosion measurements on 8x13 chain elements. The measured cross-section reductions of the chains are statistically evaluated and their results are implemented in an advanced finite element model using multilayered volume elements and stochastic analysis considering the corrosion effect in the design. The corroded chains and the numerical model are shown in Fig. 4.

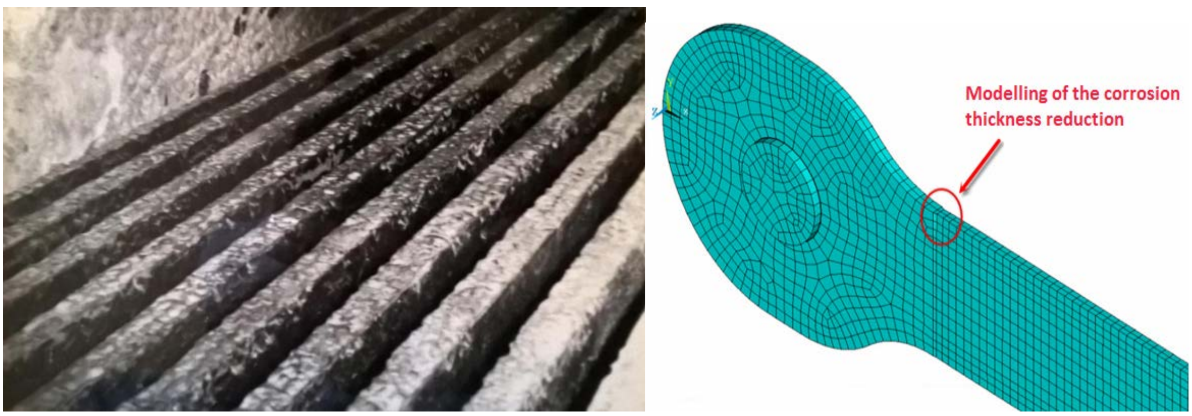

Fig. 4: Corrosion of chains and the FE modelling

Numerical models are developed for an individual chain element and for the whole chain system as well. The corrosion is considered by real surface modification of the finite elements according to the on-site measurements. The load carrying capacity of the chain is determined using numerical simulations (MNA – material nonlinear analysis) containing the corrosion. The static check is executed for pure tension force, and for combined tension force and bending moment considering the different fixation of the pins (Fig. 5). Thus the corrosion has quite a large uncertainty, therefore a stochastic model is also developed for the probabilistic design of the chain elements. Monte-Carlo analysis based probabilistic simulations are executed, where the Monte-Carlo simulation is mixed with the MN analysis. Based on the stochastic analysis the mean value and the 0.1% lower fractile (design value) of the ultimate load are calculated, which showed a maximum utilization ratio of 98% (Dunai et al., 2017).

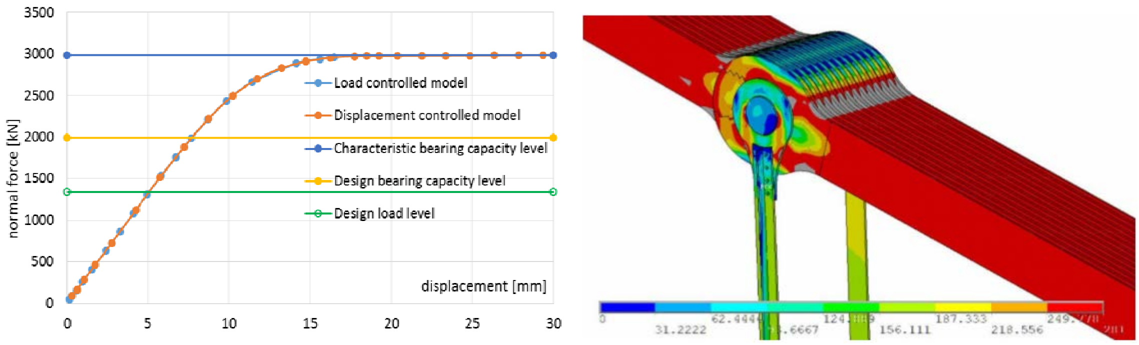

Fig. 5: Advanced analysis of corroded chains – resistance and fatigue

Another important question during the structural analysis and design of the renewal process is the condition and rotational capacity of the pins between the chain elements, because the load carrying capacity and fatigue properties of the chain elements are highly influenced by their behaviour. Previous investigations showed that stuck pins can change the structural behaviour of the bridge, as happened at the Silver Bridge leading to a progressive collapse of the bridge in 1967 in the USA. Therefore, detailed investigation and on-site tests are carried out with the aim to investigate the rotational capacity of the pins between the chain elements and to determine the change in the normal force and bending moment levels under live load. Preliminary numerical calculations (Fig. 6b) showed, if pins are stuck, significant bending moments can only develop within the chain elements at the two side of the pylons and at the abutments. These are the places where the change in the normal force and bending moment levels are measured during the test program, which have close relationship with the rotational capacity of the pins (rotation reduces or eliminates bending moment). The bridge is loaded by 12 trucks (Fig. 6a) with an average weight of ~200 kN. Trucks are placed in 13 different loading arrangements simulating partial and total loading situations. The aim of the loading and unloading was to get to rotate the pins, if they are not stuck.

The following measurements are executed on the bridge:

- deflection measurements along the longitudinal axis and actual shape of the bridge superstructure during all loading situations,
- continuous strain measurements at 8 different locations using 80 strain gauges (gauge locations are shown in Fig. 7) to determine normal stresses (normal force and bending moments separately) within the chain elements,
- influence line measurements on the chain to check the global structural behaviour,
- eigen-frequency measurements on the non-loaded bridge.

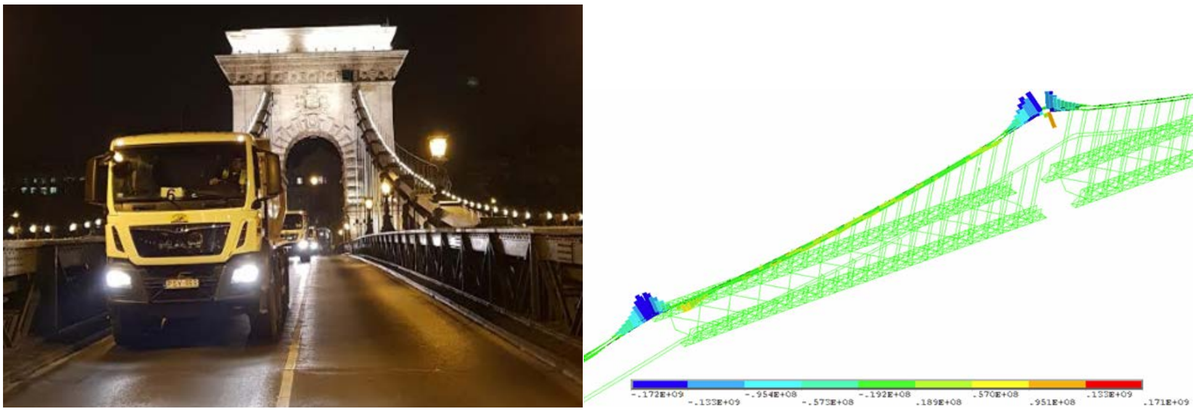

Fig. 6: Test load on the bridge and calculated changes in the bending moment diagram

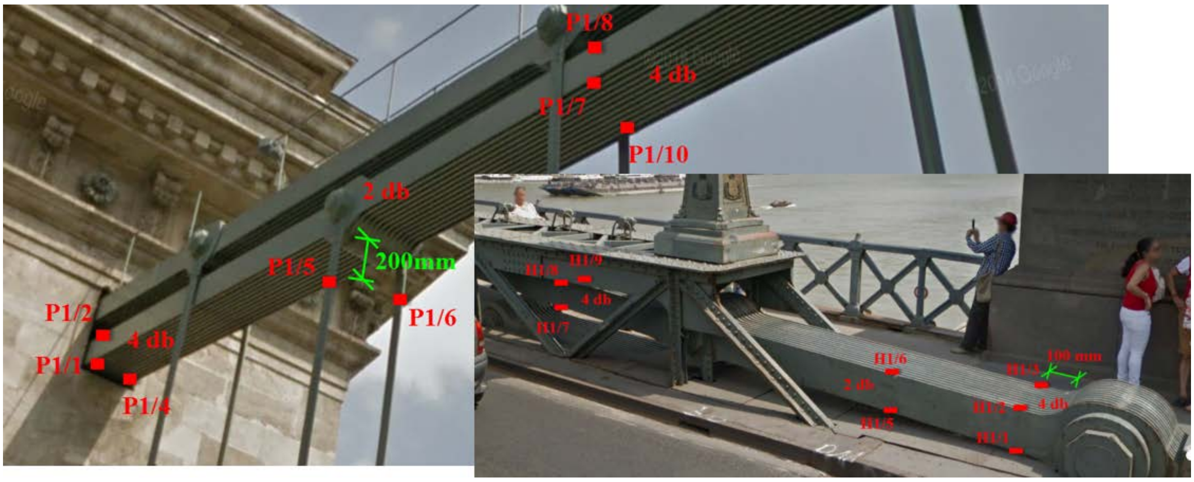

Fig. 7: Locations of the strain gauges on the chains near to the pylon and abutment

Deformation measurement results show, that the measured structural behaviour is in good agreement with the trend of the numerical calculations. No sudden slip, no remarkable rotation occurred during measurement. However, the maximum measured deflection is smaller than the numerically calculated maximum displacement, which predicted certain stucking effect at the pylons. Strain gauge measurements are compared to each other and to the numerical results as well. Numerical calculations show that only the first chain element at the pylons and abutments have significant bending moment if pins are stuck and there is only dominant normal force in all the other chain elements irrespective on the pin's rotational capacity.

The measured results prove that from the 8 measured locations only one pin rotated under the applied load level and pins are stuck at all other locations. One example for the measured normal stresses at 8 strain gauges is presented in Fig. 8. Gauges H1/1 – H1/6 are placed on the extreme fibre of the first chain element next to the abutment and others are located on the second one. Results give evidence on the developed bending moment within the first chain element which is not reduced due to the stuck of the pins. Measurement results also showed that there is a normal force level difference between the two sides of the pylon, which shows that roller supports of the chain suspension system on the top of the pylons are also stuck. The measured influence line, shown in Fig. 9, proves this observation; there are no measured strains at the left side span of the bridge. Results of the current measurement gave essential information to the designers to evaluate the load carrying capacity of the chain elements and to identify optional problems to be solved during the renewal process.

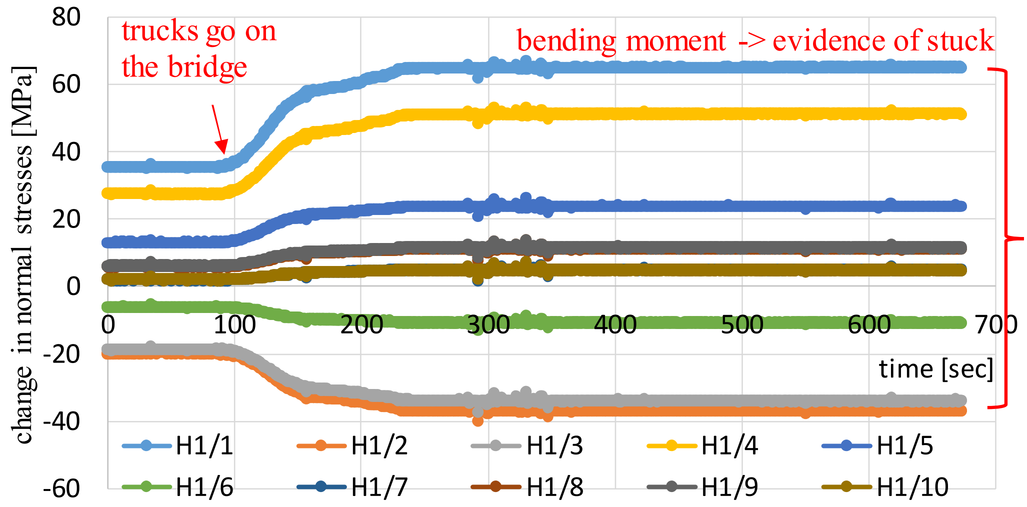

Fig. 8: Strain gauge measurements demonstrating bending moment and stuck of the pins

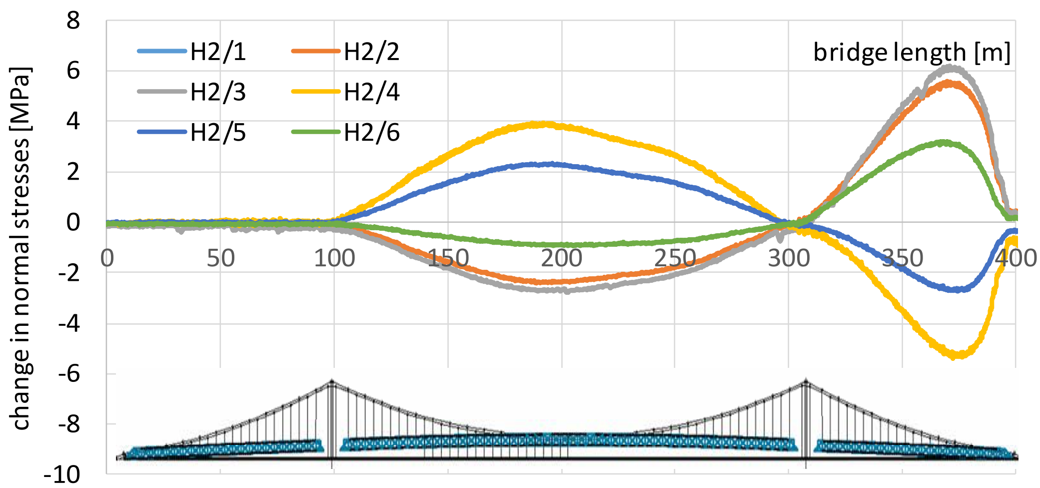

Fig. 9: Influence line of the first chain element in the side span next to the abutment

3. MARGARET BRIDGE

The original Margaret bridge was built between 1873-1876, as a 6-span-arch bridge having 6 arches in each span beside each other (Németh, 2007). The designer of the bridge was a French engineer, Ernest Gouin. The arches were closely placed from each other and connected by a quite dense stiffening system. The bridge was in operation till 1935 in the same form, as presented in Fig. 10. The first reconstruction of the bridge was made between 1935-1936. Thus the traffic volume increased on the bridge, it has to be extended by two additional arches to increase the width of the carriageway. The refurbishment was designed by Prof. Győző Mihailich. The general concept and the layout of the bridge was unchanged, the support condition of the arches, however, are replaced by pinned supports. After only one decade the bridge was exploded during the Second World War; from the 6 spans, arches of 3 spans are fallen into the Danube. The reconstruction started in 1946 and finished in 1948. The new construction had 6 larger arches in each span having a reinforced concrete deck. Newer reconstruction of the bridge has become essential in 2008 due to corrosion problems (Németh and Nagy, 2008). Thus the bridge has a specific and very important role in the Budapest public transportation, the reconstruction had to be made under traffic.

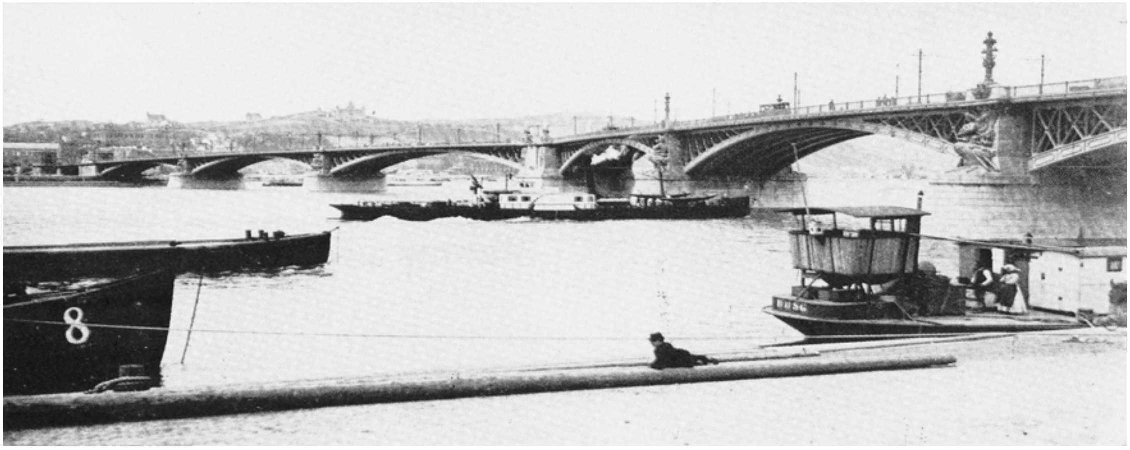

Fig. 10: The original Margaret bridge

Thus the bridge contained 8 independent arches within the cross-section in each span, it was possible to cut the deck plate in longitudinal direction into two pieces. The traffic was kept on the southern side of the bridge and the reconstruction was started on the northern side (Fig. 11). Later on the construction side was changed. The cross-section of the new bridge with orthotropic steel deck is shown in Fig. 12.

Fig. 11: Reconstruction of Margaret bridge under traffic in 2009-2011

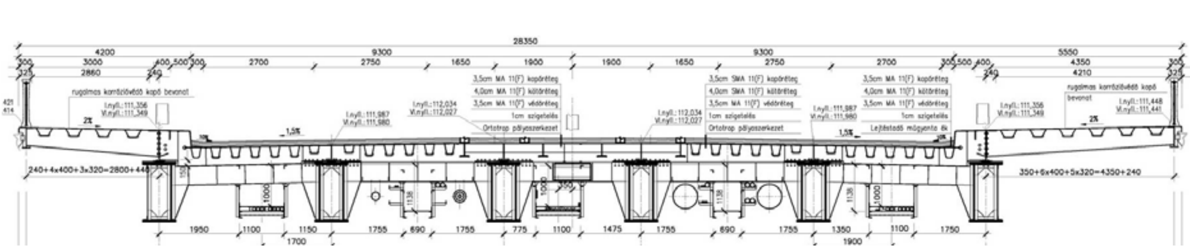

Fig. 12: New cross-section of the bridge with steel orthotropic deck

During the static check of the reconstruction it turned out that the stresses during the construction stage under the temporary tramway can be larger than their allowed stress limits. Therefore, on site measurements are executed by the Department of Structural Engineering, Budapest University of Technology and Economics on the bridge (Dunai, Jakab and Kálló, 2009). In the same time the measurements had also the aim to monitor the structure and its change in their structural behaviour during the reconstruction. The application of the monitoring system was necessary because during the cutting process in longitudinal direction the static system and the loading conditions are changed significantly. Strain measurements of the critical elements are carried out during the whole cutting process. The online monitoring system had an alarm function, if the measured stresses override their maximum allowed values. Strain measurements are executed on the stringers, stub-columns on the arches and on the cross girders. The schematic drawing of the measured structural members can be seen in Fig. 13.

The measurements finally proved that the relevant elements of the bridge behave as expected by the designers. The strain measurements, however, showed that significantly larger stress values appear in the stub-columns on the top of the arches than expected. During the refurbishment numerous fatigue cracks were detected in these elements, caused by the tram traffic. The structural behaviour of the bridge is checked by a static and dynamic loading tested after finishing the reconstruction work (Dunai, Jakab and Kálló, 2009). The loading test was executed by 24 trucks and 2 trams. The total weight of the tracks reached 7200 kN, which are placed in 27 different arrangements on the bridge. Deflection of the structure and the strains on the main girders and deck system are measured using a total of 83 strain gauges. The results of the loading test called the attention on the importance on the pier deformation consideration in the structural analysis, as shown in Fig. 14.

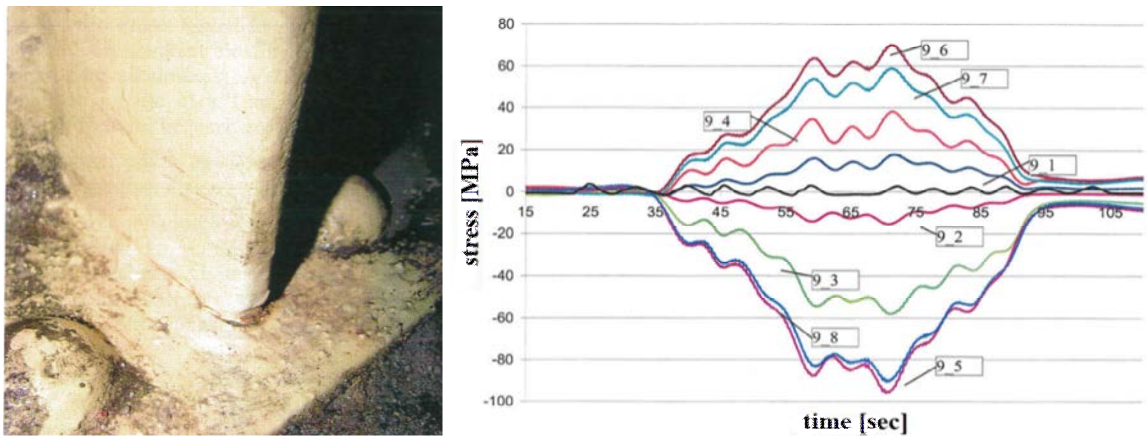

Fig. 13: Results of the on-site measurements (a, detected cracks; b, measured strains)

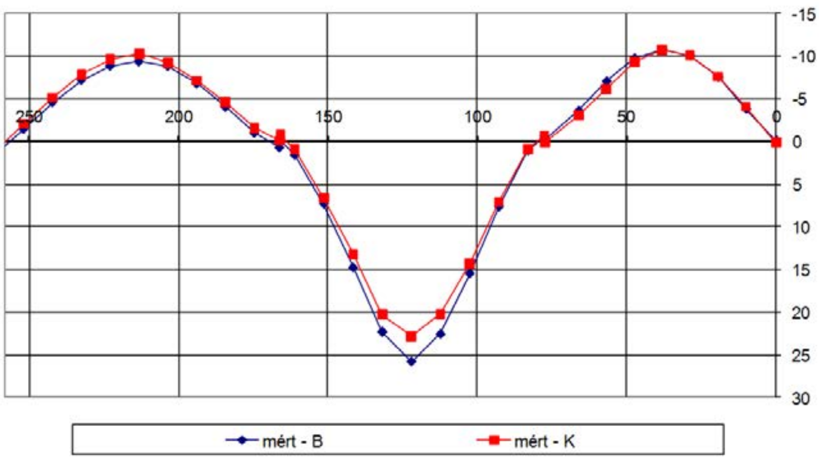

Fig. 14: Deflections under test load

The dynamic testing proved that the favourable dynamic behaviour of the bridge, the dynamic amplification factors are relatively small and the measured eigen-frequences also showed good agreement with the calculated values. The reconstructed Margaret bridge won the ECCS Steel Bridge Award in the Bridge Refurbishment category in 2012 (Dunai, Kálló, Kovács, Kövesdi and Vigh, 2009) (Fig. 15).

Fig. 15: The reconstructed Margaret bridge

4. LIBERTY BRIDGE

The Liberty bridge was erected between 1893-1896 (Gál, 2005). With its total length of 331.2 m this is the shortest Danube bridge in Budapest. The Liberty bridge is a Gerber-type truss girder which is treated as one of the most beautiful bridges of this type all over the world. The designer was the famous Hungarian bridge engineer János Feketeházy. The bridge was exploded in the Second World War in 1945 and reconstructed in 1946 in its original form. The Liberty bridge contains the largest part of their original material among the Danube bridges in Budapest. The next reconstruction on the bridge was made in the 1980's, when the bridge deck system has been completely change to a reinforced concrete deck placed on steel longitudinal stringers. In 1986 significant corrosion and fatigue problems were detected at numerous locations on the bridge, especially in the columns of the truss girder. These specific members are locally strengthened and a new bridge deck was erected under the pedestrian sidewalk. Due to the numerous corrosion problems especially on the deck system a general reconstruction of the bridge was needed in 2007 (Nagy and Horváth, 2008; Domanovszky, 2009). Supporting the design of the reconstruction a monitoring system was develop and applied on the bridge to detect the real stresses acting in the truss girder under the traffic loads by the university (Dunai, Jakab, Kálló and Kovács, 2009). The aim of the measurements was to support the designer's decision on the members to be reinforced or replace due to inappropriate fatigue life time. A typical strain time history diagram can be seen in Fig. 16.

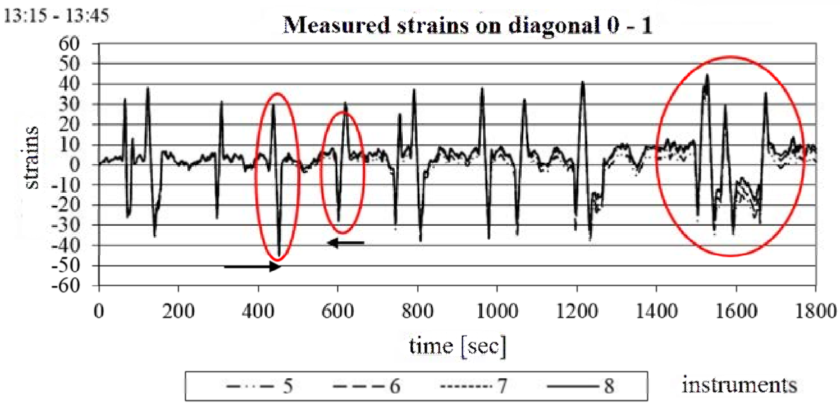

Fig. 16: Strain time history diagram

The monitoring system contained a large number of strain gauges placed on the diagonals of the truss girder, which detected the strains of the studied members under the traffic load. The monitoring system operated for several days containing weekends and working days, and produced continuous measured data rows on the stress history of the bridge. The measurements proved that the recorded stress ranges of the main girder within this time period are always smaller than the cut-off limit of the specified fatigue detail class. Therefore, the reinforcement or replacement of the main girder was unnecessary and the designers decided by the renewal of the corrosion protection of the main girders. The reconstruction work (Fig. 17) was executed between 2007–2008. The main points of the refurbishments are: (i) replacement of the longitudinal stringers of the bridge, which had huge corrosion problems, (ii) replacement of the reinforced concrete deck by a steel-concrete composite deck system, (iii) renewal of the corrosion protection, (iv) improvement of the tramway line to decrease the dynamic amplification factor on the bridge and (v) renewing of the truss girder elements and the pylon, which have significant corrosion problems. The loading test of the bridge was executed by the Department of Structural Engineering, Budapest University of Technology and Economics (Dunai, Kálló, Kövesdi and Vigh, 2009). Loading tests were executed before starting the reconstruction work in 2007 and after finishing it in 2009.

Fig. 17: Reconstruction work of the Liberty bridge in 2007-2009.

The strategies of the two loading tests are similar to ensure direct comparability for the results. Similar load patterns are used with the same number and weight for the trucks. A total of 32 trucks with a total weight of 7020 kN are used in the load testing. Fifteen different static load arrangements are tested while the deformations of the main girders and the stresses in the truss as well as in the bridge deck system are checked and compared to the calculated values. The measured deformations are also compared to the two loading test results and to an additional previous data row registered in 1986 by the previous reconstruction (Fig. 18). Dynamic behaviour of the bridge was also checked, the eigen-frequences and dynamic amplificatory factors are measured and compared to previous measurements and to computed values using advanced finite element modelling. During the loading test the correct behaviour (rotational capacity) of the pins at the Gerber truss is also measured and registered. The loading test executed in 2009 proved the correct structural behaviour of the bridge without any fixations at the moveable components and the bridge could be opened for the traffic again (Fig. 19).

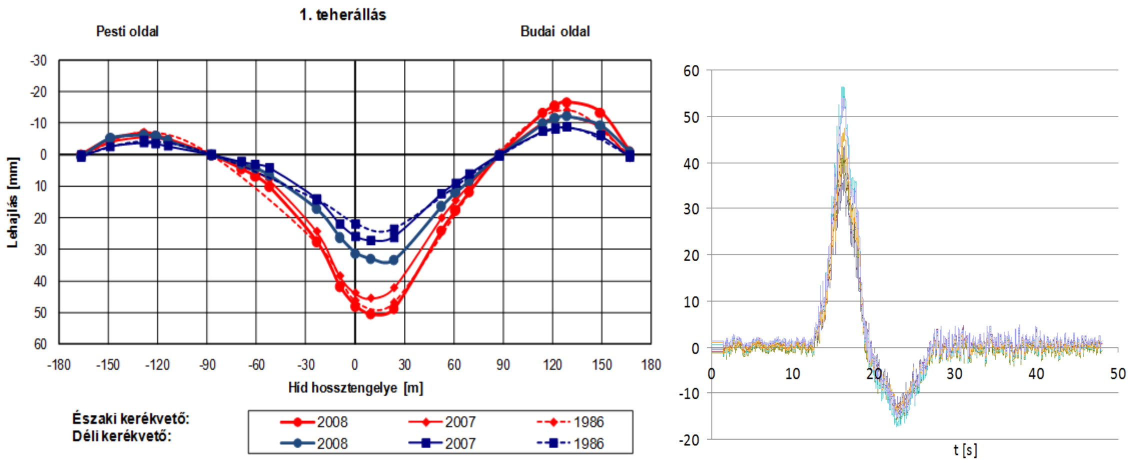

Fig. 18: Results of the static and dynamic load testing.

Fig. 19: Results of the static and dynamic load testing.

5. ACKNOWLEDGEMENT

The authors of the paper acknowledge the efforts of the whole Hungarian bridge society for their efforts on the fields of research, design, construction and maintenance to save the historical Danube bridges of Budapest for the future generations.

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EXPERIENCES OF OPERATING TEST OF TURNOUTS WITH DIFFERENT RAIL INCLINATION AND RAIL MATERIAL INSTALLED IN THE SAME RAILWAY STATION

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SUMMARY

The basic issue of railway infrastructure operators is continuously ensuring the safety and availability of the infrastructure elements at the required service level for public rail transport. Turnouts, the most frequented, sensitive and complex elements of the railway track are also affected by defects due to rolling contact fatigue (RCF).

In order to prevent or reduce the occurrence of these defects VAMAV Rail Systems LLC. a manufacturer of turnout systems has launched developments and has begun a two-year operating test together with the MÁV Hungarian State Railways Co. who is the main operator of Hungarian railway lines.

In the first half of 2017 at the same time, in the same railway station, with the same traffic load ten test turnouts were installed in order to compare their behaviour and resistance to RCF under operating conditions. The 60-1:9-300 type test turnouts have different rail inclinations (1:20, 1:40, 1:∞) and different raw materials (R260, R350HT, R400HT).

During the test period, all the inspections and necessary interventions happened in due time. The costs of these activities are gathered systematically, which can be useful for the design of operation and maintenance based on proactive Life Cycle Management. After two years operation there were no significant differences between the performances of the test turnouts. At the speed and traffic load described above, experiences show that in case of regular inspection and proper maintenance all of them behave well.

1. INTRODUCTION

Operators, who are providing railway services (passenger transport, freight transport, infrastructure operation) have a contractual and legal obligation for continuously ensuring the safety and availability of the infrastructure elements at the required service level. Among other issues, they must provide the proper operation and function of the railway infrastructure, the establishment of the conditions of safety of life and property, the technical supervision and the maintenance.

Turnouts are the most frequented, sensitive and complex elements of the railway track. In this case the careful designing, material selection, geometrical and structural design, proper installation and effective conservation can be achieved by taking the necessary measurements and interventions in due time.

Defects due to rolling contact fatigue (RCF) occurring on railway network of MÁV Hungarian State Railways Co. (in the following: MÁV) also affect turnouts.

In order to prevent or reduce the occurrence of these defects VAMAV LLC. (in the following: VAMAV) a manufacturer of turnout systems has launched developments that affect the geometry of structural elements, the quality of the used materials and the manufacturing technology.

VAMAV presented the developments to MÁV in order to observe the test structures under operating conditions. The information and results generated during the operating test can be used in the future both in the design and in applicability of turnouts. The lessons learned may be further useful for the design of operation and maintenance based on Proactive Life Cycle Management. They can be useful as a basis for deciding the technical content and parameters of future turnouts also.

Based on the request of VAMAV, the railway operator MÁV designated a test location (see Fig. 1) for the installation of test turnouts. The axle load of the railway line is 225 kN and the traffic load is approximately 8 million gross tons/year/track.

In the first half of 2017, ten pieces of turnouts were installed. The 60-1:9-300 type test turnouts (see Fig. 4) have different rail inclinations (1:20, 1:40, 1:∞) and different raw materials (R260, R350HT, R400HT). The operating test is based on an "Inspection and Maintenance Agreement". The duration of the test period is minimum two calendar years and the turnouts are under monitoring at regular intervals (monthly) according to a test and evaluation protocol.

As a Customer MÁV has requested that the turnouts to be installed must have such a structural design that in case of malfunctioning or disrupting life or property safety, the main parts must be replaced from the warehouse of MÁV or directly from the factory, at least temporarily, within 48 hours with standard elements. The installed turnouts fulfil this requirement.

2. LOCATION AND CIRCUMSTANCES OF THE OPERATING TEST

- Test location: Tápiógyörgye railway station, MÁV Nr. 120a Rákos - Szolnok railway line.
- Permissible speed of the turnouts: 160 km/h in main line, 40 km/h in branch line.
- Actual speed of the turnouts: 120 km/h in main line, 40 km/h in branch line.
- Axle load: 225 kN.
- Traffic load (gross tons / year): right track 8.66 million and left track 7.95 million.

The Operator (MÁV) together with the Manufacturer (VAMAV) based on the "Inspection and Maintenance Agreement" performs operating test of turnouts. Fig. 2 shows the parameters of the test and Fig. 3 shows the total traffic load of the turnouts since installation.

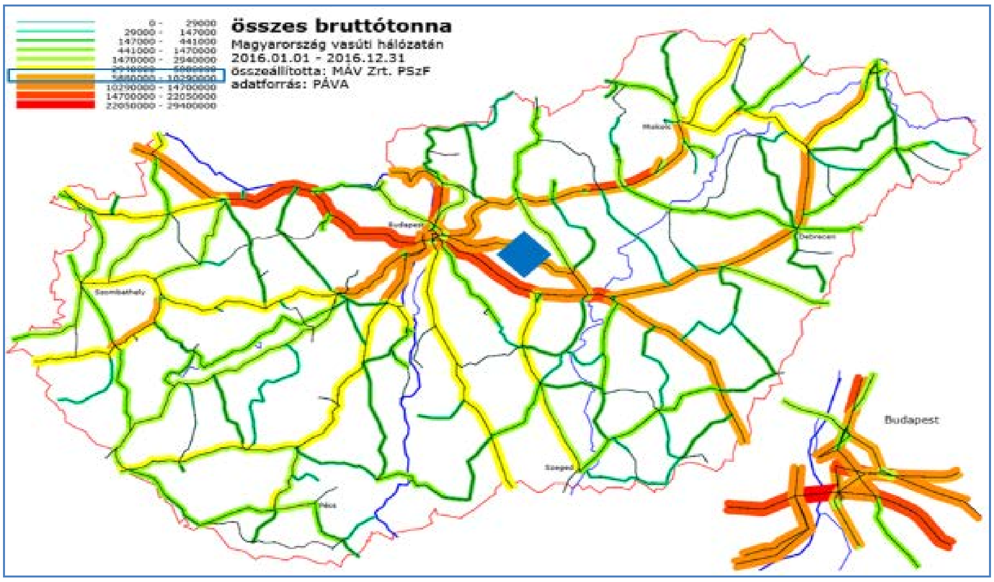
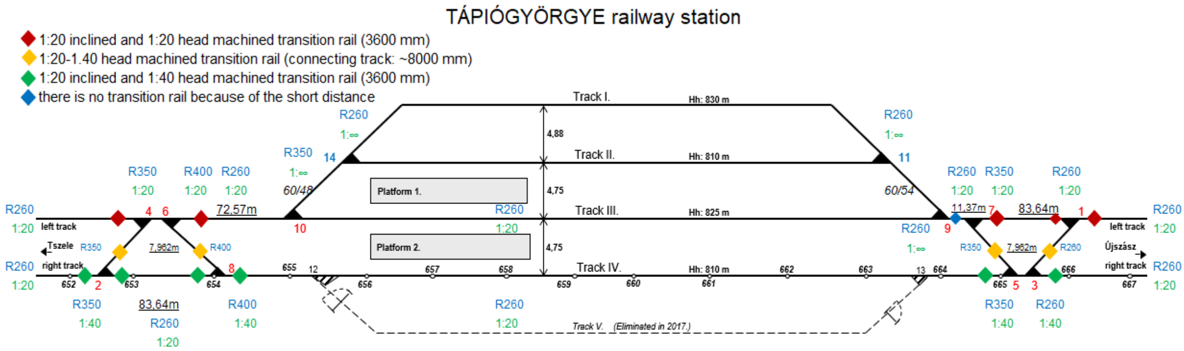


Fig. 1: Tápiógyörgye railway station in Hungary



Turnout Nr.	1	2	3	4	5	6	7	8	9	10
Installation	2017.08.18	2017.03.31	2017.03.11	2017.04.10	2017.03.11	2017.04.10	2017.07.13	2017.03.31	2017.07.14	2017.06.08

Fig. 2: Test parameters

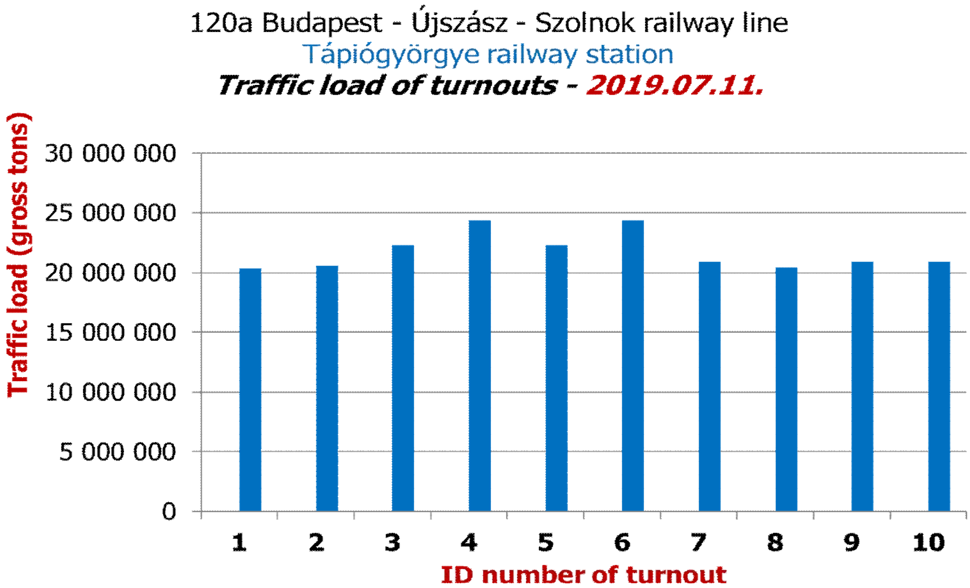


Fig. 3: Traffic load of turnouts

3. TECHNICAL PARAMETERS OF TEST TURNOUTS

Fig. 4 shows the layout plan and Tab. 1 and 2 contains the main geometric and structural parameters of the test turnouts. They have welded rail joints, concrete sleepers (without under sleeper pads), SKL-3 (Vossloh) fastenings and ribbed baseplates. In the switch panel, some baseplates have VMGÖ type roller (VAMAV). The inner fastening of stock rails is IBAV. The locking device of the turnouts is SPHEROLOCK (voestalpine) in steel hollow sleeper.

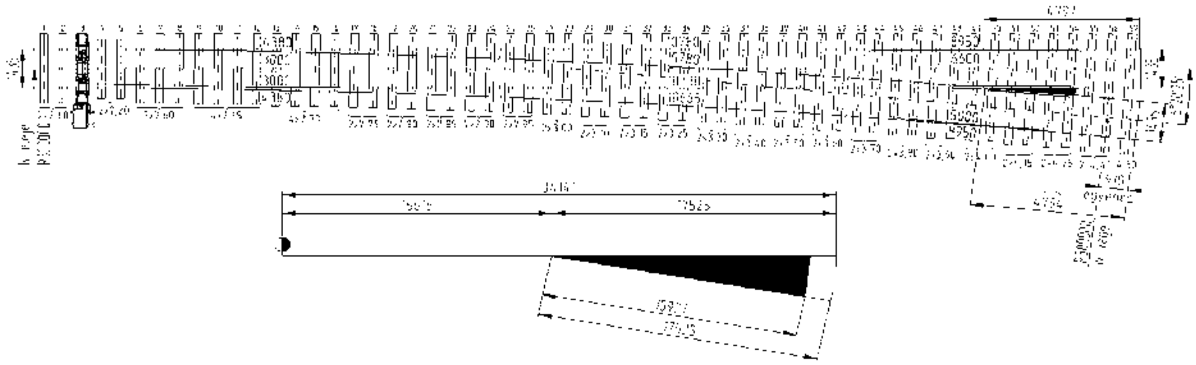


Fig. 4: Layout plan of VAMAV 60-1:9-R300 turnout

Tab. 1: Geometric parameters of VAMAV 60-1:9-R300 turnout

Parameters	
Tangent of the angle	1:9
Radius	300 m
Gauge	1435 mm
Design speed	160 km/h (main line), 40 km/h (branch line)

Tab. 2: Structural parameters of VAMAV 60-1:9-R300 turnout

Part	Raw material	Profile before machining	Inclination
Switch rails	R260, R350HT, R400HT	60E1A1	1:20
Stock rails Intermediate rails Closure rails Outside rails	R260, R350HT, R400HT	60E2	1:20 1:40
Crossing block	Mn13 (Gx120Mn12)	special	1:∞
Check rails	R260, R320Cr	33C1	1:∞

3.1. Inclination of the turnouts and the transition zones

If one of the turnouts is directly connected to another turnout than turnouts without rail inclination or turnouts with the same rail inclination can be used. Where the rail profile of the turnout differs from the rail profile of the connecting track a transition zone is necessary in the connecting track. In order to meet the requirements for interchangeability the inclination of the rails in the turnouts was not determined by inclination of the rail foot and rail web but by the machining of the railhead (milling). This way it was unnecessary to reconstruct the concrete sleepers and the baseplates. Special transition rails (twisted rails, milling of the railhead) and using standard transition concrete sleepers have achieved the transition of the various rail inclinations between the different turnouts and between the turnouts and the connecting track (1:100, 1:50, 1:33, 1:25). In the operating test, 1:20 rail inclination has been applied in the turnouts installed in the left track and 1:40 rail inclination in the turnouts installed in the right track of the railway station.

3.2. Pre-assembly, transportation and installation

The test turnouts were pre-assembled in the factory that ensures the excellent dimensions. Transportation happened by rail. Just in time transportation (JIT) enables the fast installation and needs no depots on site. The main parts of the turnouts (switch panel, closure panel, crossing panel) arrived at installation site fixed onto sleepers and the additional parts were set.

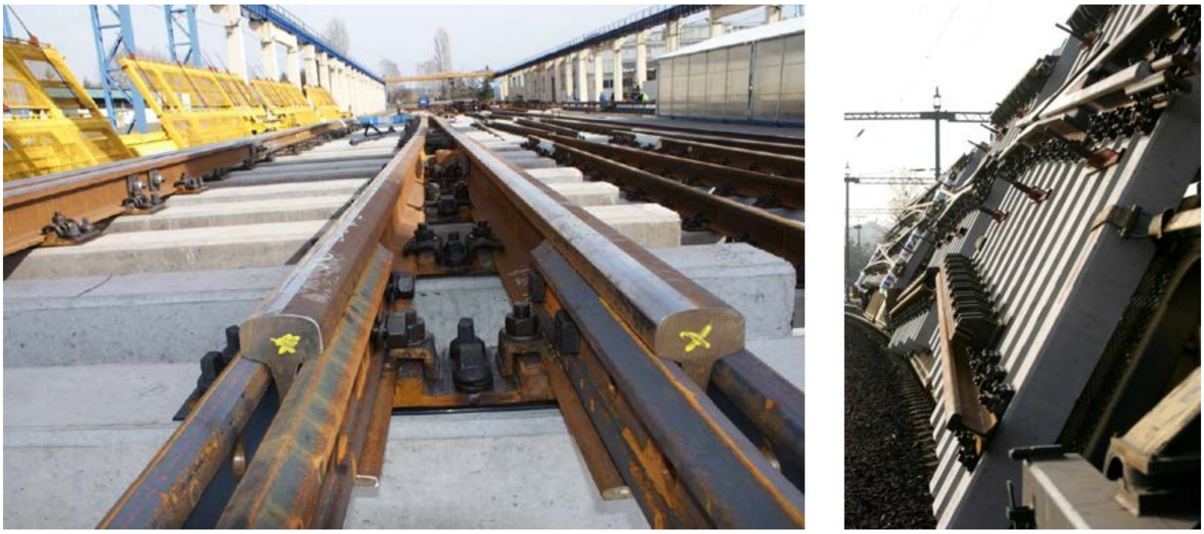


Fig. 5: Pre-assembly and transportation

Installation on site were done by a KIROW-crane.

Fig. 6: Installation on site

Fig. 7: The new turnouts

4. INSPECTION METHODS AND RESULTS

The experts of MÁV and VAMAV check the status of the turnouts monthly. Different basic and additional tests and measurements have been taken according to the internal regulations of MÁV. Here can be seen all the tests, measurements and their result.

- Obstacle test / object detection (according to the internal regulations of MÁV 103140/1989): During the periodic inspections, the locking devices were in a fault free state for all turnouts. At a 2 mm thick obstacle an end position signal can be seen, and at a 4 mm thick obstacle the end position signal cannot be seen.
- Locking device check (according to the internal regulations of MÁV D.5.): During the periodic inspections, the operating dimensions and function performance of the locking devices were correct for all turnouts.
- Opening at the drive position (according to the internal regulations of MÁV 103140/1989): Nominal value is 160 mm. During the periodic inspections, the values of opening at the drive position for all turnouts were within the intervention limits ($\pm$5 mm).
- Longitudinal displacement of stock rail and switch rail (according to the internal regulations of MÁV D.5.): Nominal value is 0 mm. During the periodic inspections, the positions of the checkpoints (made by the factory at production) were appropriate for all turnouts.
- Force measurement (according to the internal regulations of MÁV 103140/1989): Nominal value of actuation force is 2.2 kN, actuator capacity is 3 kN, trailing force is between 6-12 kN and the force that remains in the rod because of the bending strength of the open switch rail is 0.8 kN. During the periodic tests, the actuation forces were appropriate for all turnouts.
- ORE derailment safety test (according to the internal regulations of MÁV D.54.): During the periodic tests, the wear of the beginning of stock rails and switch toes did not exceed the allowed values for one of the turnouts.
- Gauge (according to the internal regulations of MÁV D.54.): Nominal value is 1435 mm. During the periodic inspections, the values of gauge for all turnouts were within the intervention limit (-3 mm / +5 mm).

- Cant / superelevation (according to the internal regulations of MÁV D.54.): Nominal value is 0 mm. During the periodic tests, the values of cant for all turnouts were within the intervention limit (±6 mm).
- Width of the free wheel passage area (according to the internal regulations of MÁV 103140/1989): Nominal value is maximum 1371 mm. During the periodic inspections, the values of this parameter for all turnouts were under the maximum value allowed during maintenance (1376 mm).
- Check gauge (according to the internal regulations of MÁV 103140/1989): Nominal value is 1394 mm. During the periodic inspections, the values of the check gauge for all turnouts were within the intervention limits (±2 mm).
- Profile check (wear) (according to the internal regulations of MÁV D.54.): Nominal value is 0 mm. During maintenance the maximum value of height wear of switch rail is 4 mm, side wear of switch rail is 5 mm, height wear of crossing block is 4 mm and valley of the surface of crossing block is 2 mm.

  During the periodic inspections, different cross-sections were measured in the switch panel according to the following figure (C1 = 2526 mm, D = 3299 mm, E = 4395 mm, from the switch toe). The average wear is less than 0.8 mm (approximately at the cross-section C1). The smallest wears occurred in the turnouts Nr.6, Nr.7, Nr.8. Turnouts Nr.6 and Nr.8 have been installed 3 months earlier than turnout Nr.7 and their traffic load is more with 3 million tons.

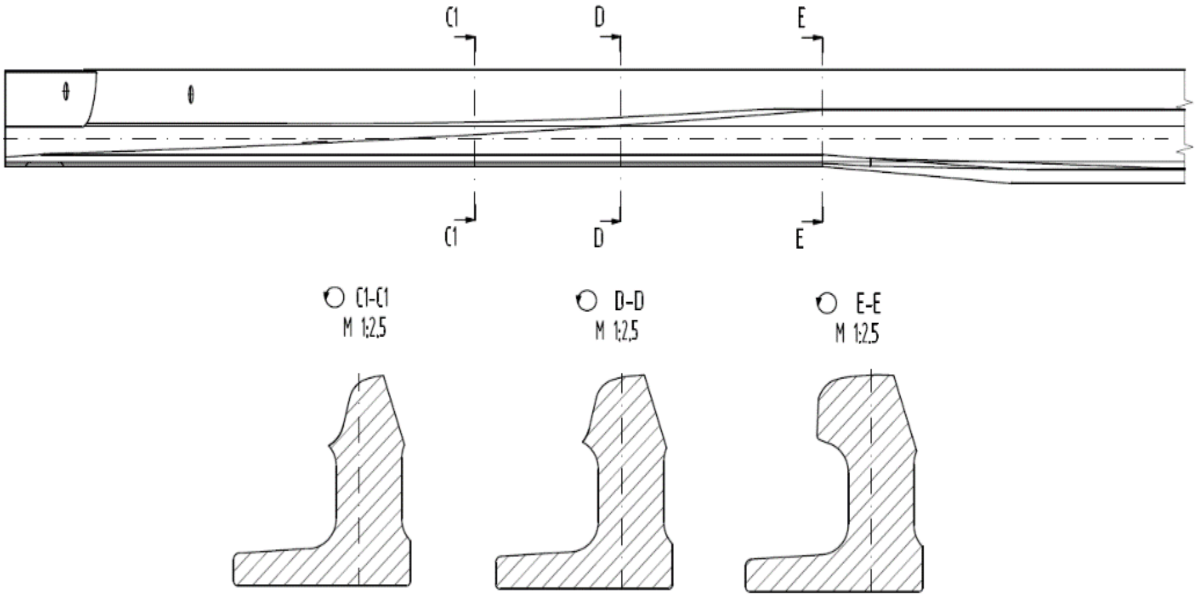


Fig. 8: Profile measurement points in switch panel

During the periodic inspections, different cross-sections were measured in the crossing panel according to the following figure (E = 204 mm, EKP+500 = 3299 mm, F = 727 mm, from the theoretical intersection point). The maximum wear is 2.3 mm and the average wear is less than 1.5 mm. The smallest wears occurred in the turnouts Nr.1 and Nr.4. The installation of turnout Nr.4 happened 4 months earlier than turnout Nr.1 and its traffic load is more with 4 million tons.

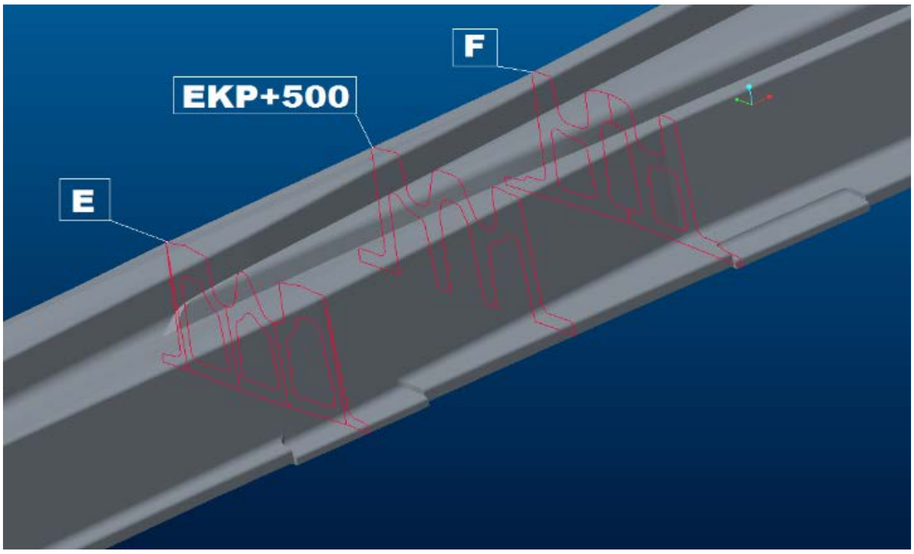


Fig. 9: Profile measurement points in crossing panel

- Hardness (MSZ EN 13674-1): During the periodic tests, it was found that the initial surface hardness of the raw materials did not get worse. There is a kind of hardening as the result of traffic load.
- Penetration test (MSZ EN 473): During the periodic inspections we detected twice (March, October 2018.) small HC faults in a short 100-150 mm length in the turnout Nr.10 (1: ∞, R350HT). In both cases, HC appeared on the straight switch rail where the vehicle's wheel overloads from the stock rail to the switch rail (see Fig. 10). This location in the case of R=300m turnouts is approximately ~4000 mm from the switch toe (total length of switch rail is 13000 mm), above the sleeper Nr.9. (see Fig. 4).

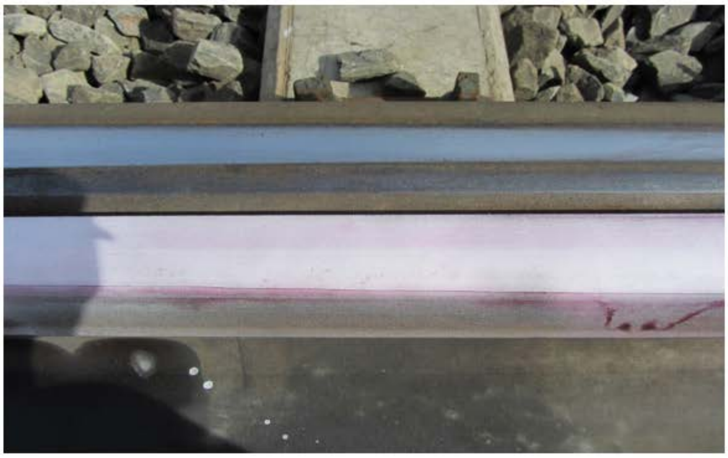


Fig. 10: HC fault in turnout Nr.10

During the periodic inspections it has been detected once (December 2018.) a small crack (60-80 mm long and 0.5 mm deep) in the turnout Nr.8 (1:40, R400HT) on the crossing nose (see Fig. 11).

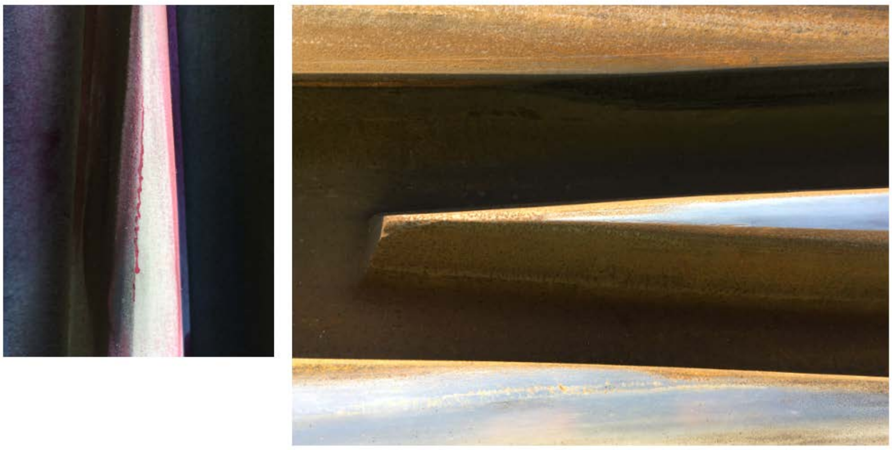


Fig. 11: Crack in turnout Nr.8.

- Eddy current test (according to the internal regulations of MÁV D.10.): Nominal value of damage depth (mk) = 0 mm. Maximum value during maintenance: "A" (mk < 0.5 mm) or "B" (0.5 mm < mk < 1.5 mm) category. During the periodic inspections, the eddy current measurements did not detect any HC fault in one of the turnouts except for Nr.10. In case of the turnout Nr.10 the HC faults, discovered by the liquid penetration tests, were confirmed by the eddy current tests too (fault categories “A” and “B”).

5. MAINTANENCE WORKS DONE

During the previous two years of the operating test the daily operation issues (inspection, cleaning of the switch device) happened periodically. The defects and failures, which were found during the inspection, have been eliminated with the frequency corresponding to the technical necessity (see Tab. 3). To eliminate the defects and failures mentioned above maintenance works happened three times, September 2017, June 2018, February 2019 (adjustment of roller system, checking of locking device, grinding and HC remover grinding, crack repair with filler welding) (see Fig. 12 and 13). The first one was the so-called "First Maintenance" which contains the adjustment of rollers, checking of locking device and rail profile grinding. There were three times tamping, which were guarantee regulations, related to the installation of turnouts.

Tab. 3: Inspection and maintenance works

Inspection / Maintenance work	Resources
Inspection	5 persons x 29 days x 8 hours, car
First maintenance 2,3,4,5,6,7,8,9,10 turnouts September 2017. (grinding)	2 persons x 2 days x 8 hours, car, machine, excipient
Tamping machine October 2017.	2 x 8 hours
Tamping machine March 2018.	2 x 8 hours
Second maintenance 1,5,6,9,10 turnouts June 2018. (grinding, weld up repair)	2 persons x 2 days x 8 hours, car, machine, excipient
Tamping machine November 2018.	2 x 8 hours
Third maintenance 3,5,8,9,10 turnouts February 2019. (grinding, weld up repair)	3 persons x 1 days x 8 hours, car, machine, excipient

Fig. 12: HC grinding in turnout Nr.10.

Fig. 13: Crack repair with filler welding in turnout Nr.8.

6. LIFE CYCLE COSTS (LCC) – LIFE CYCLE MANAGEMENT (LCM)

Optimal (both technically and economically) use of limited available resources and minimizing the operating and maintenance costs during the life cycle (LCC) can be achieved by the modernization of infrastructure elements and their monitoring and maintenance system. One of the possible methods can be the technical developing of the structures (e.g. turnouts) and the expansion of existing maintenance system with a proactive Life Cycle Management (LCM). It means that if the initial high quality of the infrastructure elements and their proper maintenance is provided, the total costs during the life cycle can be lower. Over the financial benefits, the main advantages of the system are the realization of full range of maintenance strategy and the met of RAMS principles. They are the next: R= Reliability, A= Availability, M= Maintainability and S= Safety.

During the operation test, the costs of the installation, inspection and maintenance activities are gathered systematically in a so-called LCM table for each turnouts (see Fig. 14). As you can see, the tables contain the traffic circumstances (axle load), the technical parameters of the turnouts, and the types of activities (installation, inspection, maintenance and reinstallation) which can occur during the lifetime, the unit prices and the quantity of them per year. The total costs of works per year, the net present value (NPV) and the future value (FV) appears at the bottom line of the table.

LCM PROTOKOLL ID	TGY03
Permissible speed (main / branch line, km/h)	160 / 40
Axle load (kN)	225
Traffic load (tons / year / track)	8 663 413
Number of tracks of railway line	2
Classification of T/O	R > 150m
Track system (welded / fish plated)	welded
Type of T/O (rail-angle-radius)	60-1:9-300
Rail quality	R260
Type of fastenings	SKL-3
Type of sleepers	concrete
Type of ballast	crushed stone
Type of protective - reinforcing layer	sandy stone
Subsoil	medium clay
Planned lifetime (years)	30
Examined quantity (T/O)	1

Type of maintenance working	Unit	Unit price	0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31
INSTALLATION	T/O	"	1																															
--- Inspection ---	hours	"		400	480	280																												
--- First maintenance (grinding) ---	m	"		1																														
--- Ballast tamping ---	m	"		1	2																													
--- Grinding ---	m	"			1	1																												
--- Weld up repair ---	piece	"			1	1																												
--- Maintenance working 5 ---	piece	"																																
--- Maintenance working 6 ---	piece	"																																
--- Maintenance working 7 ---	m3	"																																
REINSTALLATION	T/O	"																																1
Costs per year			"	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"
Calculated value of costs (NPV, FV)			"	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"
Life cycle costs			"																															

Fig. 14: LCM table for turnout Nr.3.

At this moment, all the combination of material and inclination is performing well and there is not big differences between their inspection and maintenance costs. In the future, the systematically gathered data and experiences can be quite useful for the design of operation and maintenance based on proactive Life Cycle Management.

7. CONCLUSIONS OF THE OPERATING TEST UP TO NOW

In order to prevent or reduce the occurrence of the RCF defects VAMAV Rail Systems LLC. a manufacturer of turnout systems has launched developments and has begun an operating test together with the MÁV Hungarian State Railways Co. In the first half of 2017 ten pieces of test turnouts were installed with different rail inclinations (1:20, 1:40, 1:∞) and different raw materials (R260, R350HT, R400HT) in order to compare their behaviour and resistance to RCF under operating conditions. The location of the test is Tápiógyörgye railway station on the Nr.120a Rákos - Szolnok railway line. The turnouts are under monitoring at regular intervals (monthly) according to a test and evaluation protocol. After two years operation we have concluded the next:

- Regular, objective diagnostics and "First Maintenance" are important (approximately after 4 million gross tons traffic load).
- Several factors (substructure, superstructure, geometry) affect the state of the turnouts.
- The initial hardness of the surfaces did not get worse.
- The wear is below 2.3 mm during ~ 2 years of operation.
- During the test period minimal steel part maintenance (grinding, welding) and some ballast tamping happened.
- HC faults in turnout Nr.10 (1: ∞, R350HT) occurred twice (9 months after the installing of the turnout and another 7 months after the first appearance of HC). In the other turnouts, there are no RCF-type faults.
- A key issue is the professional intervention in due time.

After two years operation there were no significant differences between the performances of the test turnouts. At the speed of 160 km/h in main line and 40 km/h in branch line and at approximately 8 million gross tons / year traffic load, experiences show that in case of regular inspection and proper maintenance all of the test turnouts behave well. Therefore, the operating test is suggested to be continued for another two or more years to see significant differences between the combinations.

THE PAST, PRESENT AND FUTURE OF TUNNEL CONSTRUCTION IN HUNGARY IN THE LIGHT OF INTERNATIONAL PRACTICE

A MAGYARORSZÁGI ALAGÚTÉPÍTÉS MÚLTJA, JELENE, JÖVŐJE A NEMZETKÖZI GYAKORLAT TÜKRÉBEN

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SUMMARY

For thousands of years, humanity has been building linear underground structures for transportation, storage, or other special purposes. In Hungary, the construction of the tunnels still in operation today began in the 1850s. First the Várhegy (Castle Hill) tunnel was built between 1853 and 1857, which was followed by the construction of the railway tunnel between Kis-Gellérthegy and the Southern Railway Station in 1861. Between 1895 and 1911, 8 shorter (96 m) and longer (780 m) railway tunnels were built. Contemporary road tunnel construction in Hungary started with the construction of a tunnel chain consisting of 4 twin tunnels on the M6 motorway between Bátaszék and Véménd.

ÖSSZEFOGLALÓ

Az emberiség már több évezred óta épít közlekedésre, szállításra, tárolásra vagy egyéb különleges célra szolgáló, a föld felszíne alatt elhelyezkedő vonalas jellegű építményeket. Magyarországon a ma is üzemben lévő alagutak létesítése az 1850 években indult el. Elsőnek a Várhegyi alagút épült meg 1853 és 1857. között, ezt követően épült meg a Kis-Gellérthegy – Déli pályaudvar - vasúti alagút 1861.-ben. Az 1895. és 1911. évek között további 8 rövidebb (96 m) és hosszabb (780 m) vasúti alagutak létesültek. Magyarország jelenkori közúti alagútépítése az M6 autópálya Bátaszék-Vémend közötti szakaszán megépült 4 db ikerjáratos alagútból álló alagútlánc megvalósulásával kezdődött.

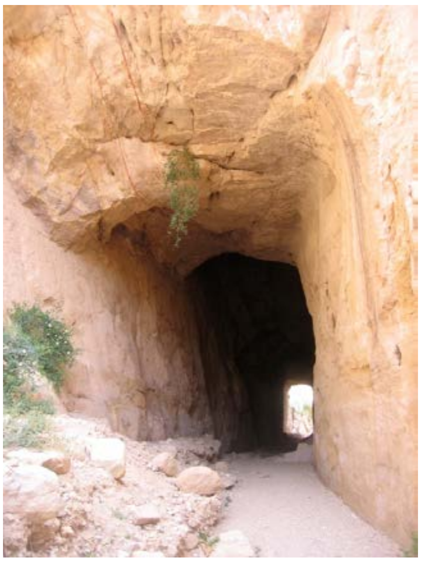
1. A KEZDETEK

Az első közlekedési célokat szolgáló alagút Babilonban az Eufrátesz medre alatt épült i.e. 2180-60 között, amely 4,5 m széles, 3,6 m magas szelvénnyel és közel 1 km-es hosszal a királyi palotát a templommal kötötte össze. Az építés idejére a folyót elterelték, az alagutat nyílt munkaárokban bitumenhabarcsba rakott téglaboltozattal építették meg.

Jeruzsálem Óvárosában i.e. 700 körül épült a Siloam-alagút, amely 500 m hosszával vizet szállított és mai is működik. Az i.e. 200 és i. sz. 100 között virágkorát élő Petrában – a nabateusok fővárosában – árvízvédelmi célokkal építették meg a 86 m hosszú Nabataeans water tunnel elvezető alagutat, amely a Siq-et - Petrába bevezető 70 m mély és kb. 1,5 km hosszú kőzethasadékot - mentette meg az időszaki áradásoktól.

1. Ábra: Siloam-alagút (Wikipedia)

2. Ábra: Nabataeans water tunnel



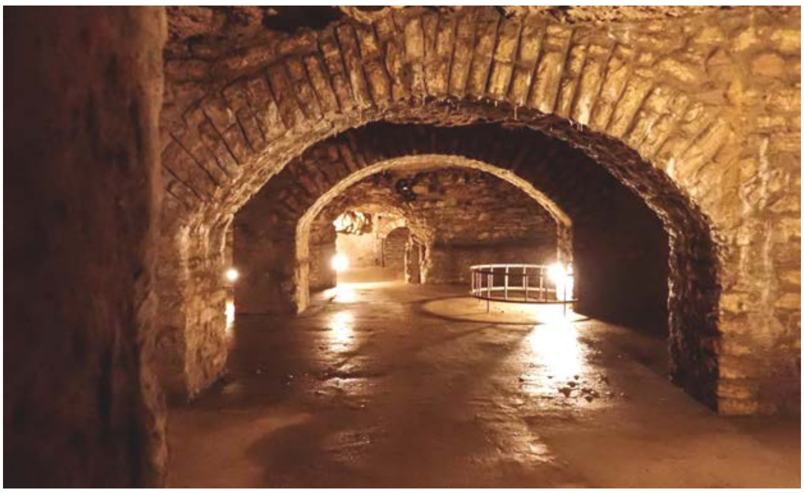
3. Ábra: Nabataeans water tunnel

Az első közúti alagútnak nevezhető felszín alatti létesítményt kb. 2000 évvel ezelőtt, Augusztus császár idejében Nápoly-Posillipóban épült a 1000 lépés hosszú és 7,5 m széles alagút a jól fejthető puha vulkáni tufában.

4. Ábra: Nápoly-Posillipó alagút

2. VÁRALAGUTAK

A középkorból csak váralagutak ismertek, amelyek a főtoronyból, pincéből, szobákból, vagy a várkútból vezettek ki a szabadba. A kútból indult ki a váralagút pl. Plessen, Dilsburg, Wurmberg, Hradek, Friedland, Elbogen, illetve Körmöcbánya, Zólyomlipcse, Murány, Füzér várainál. IV. Henrik alagúton keresztül szökött meg Harzburg várából. Turenne francia tábornok 1674-ben a váralagúton keresztül jutott be Auerberg várába. Az 1400-as években kezdték el építeni Selmecbányán a Bíber-altárót, mely 5,6 km hosszúságával a középkor legnagyobb vízelvezető alagútja volt. Különböző járatok épültek az egri vár, illetve a budai Vár alatt is. Az utóbbi ma már turisztikai látványosság.



5. Ábra: Budai Vár alatti turisztikai járat

3. ALAGÚTÉPÍTÉS MÚLTJA

A mai is tartó alagútépítési korszak az újkorban kezdődött a Temze alatt átvezető Thames Tunnel megépítésével, amely London két városrészének összekötését szolgálta. Ennek megépítését már 1798-ban, majd később 1805-1808-ban hasztalan kísérelték meg. Aztán 1825-ben Marc Isambard Brunel ismét nekilátott az építésnek. A 11 m széles, 7 m magas (az űrszelvény szélessége 4,27 m, magassága 5,18 m) és 406 m hosszú alagút 18 év alatt készült el. Ez idő alatt 16 alkalommal kellett a munkákat megszakítani elemi események – vízbetörések - miatt, míg végre 1843. március 25-én Isambard Kingdom Brunnel (Marc fia) az építést befejezte. A kiadások összege mintegy 5 millió forint volt. A létesítmény a Temze szintje alatt 23 méterre húzódik, egy helyen mindössze négy méterrel halad a meder alatt. Az átadás napján ötvenezer ember sétált át a Temze túlsó partjára, az első tíz héten pedig egymillióan tették meg ezt az utat fejenként egy pennyért. Londonnak akkor kétmillió lakosa volt.

6. Ábra: Az alagút 2005-ben az East London line része

Az alagutak építésénél tapasztalható bámulatos haladás valójában a vasutak óriási fejlődésének volt köszönhető. A mozdony erejének kellő kihasználása végett egyfelől az emelkedési viszonyok egyenletességére kellett törekedni, másfelől pedig ügyelni kellett arra, hogy az emelkedések és esések bizonyos megengedhető mértéket meg ne haladjanak. A vízválasztókon való átkeléseknél azonban e kettős cél csak hosszú vonalfejlesztéssel volt elérhető, ami az építést nagyon megdrágította. A vonalak megrövidítése érdekében tehát alagutakat építettek, és azokat a cél elérése végett nemcsak egyenesen, vagy egyszerű kanyarulatokban tették, hanem visszatérő és egymás fölött haladó spirális alakban is.

A vasúti közlekedés megjelenésével közel egy időben megépültek az első vasúti alagutak. Ilyen a London-Bristol vasútvonalon, Bath várostól keletre Isambard Brunel vezetésével az 1830-as évek második felében megépült Box-tunnel. Az alagút 2,9 kilométer hosszú és 9,1 m széles. A Box-alagutat úgy tervezték, hogy elférjen benne kettő, a Great Western Railway által használt széles nyomtávú sínpár. A munkálatok 1836 szeptemberében kezdődtek, amikor hat aknát vájtak függőlegesen a dombba. Ezek 8,5 méter átmérőjűek voltak, a legnagyobb 88 méter mélyre nyúlt a felszínről. Az eltérő kőzettípusok miatt a keleti szakaszon robbantották a sziklákat, majd az alagutat gótikus ívűre faragták. A nyugati részen a csákány és lapát is elég volt az építkezéshez. Az alagút falazatát ezen a szakaszon téglával rakták ki. Az építkezés során több mint 30 millió téglát használtak fel. A kitermelt köveket lovak erejével emelték ki az aknákon keresztül a felszínre. A munkások napi 24 órában, gyertyafény mellett dolgoztak, és kizárólag saját és a munkába fogott lovak erejére támaszkodhattak. A keleti szakaszt két irányból építették. Amikor a két szakasz összeért, csak öt centiméter volt az eltérés.

7. Ábra: Téglás szakasz és a faragott rész találkozása

Az alagutak szempontjából a 19. század egyik legérdekesebb vállalkozása az 1882. júliusában megnyitott Gotthárd-vasútvonal volt, amelynek a 206 kilométer hosszú fővonalában Immenseetől Chiassóig 65 alagút épült. Ezeknek összes hossza 42 kilométer, ebből 12 alagút egy kilométernél hosszabb. A vasútvonal korának műszaki bravúrja volt a rengeteg alagúttal, spirál-alagúttal és híddal. A Gotthárd vasúti alagút sokáig világrekorder volt a hosszával és a költségeivel A másik rendkívüli vasúti alagút a francia és olasz kormányok által 1898-1906 között épített 20 km hosszú Simplon-alagút volt.

Számos különleges vasúti alagutat építettek vízfolyások, tengeröblök alatt való áthaladás céljából is, ott, ahol a hidak építését a rossz alapozás, vagy a tengeri hajók áthaladása végett szükséges nagy magasságok akadályozták meg. Ilyen volt Dél-Wales és Anglia között, a Severn-öböl alatt épített alagút, amely 1873-1885 között épült. Hossza 7250 m, az űrszelvény szélessége 9 méter. Az alagút felett levő földréteg legkisebb vastagsága 13,5 méter. A vízoszlop magassága apály idején 18, dagálykor 30 m. Technikailag igen nevezetes volt továbbá a Hudson folyó alatt, New York és New Jersey között épített összeköttetés, mely két egymás mellé épített 4,9 m széles 5,5 m magas alagútból áll (mindegyik egy-egy vágány számára). Hossza 1670 méter.

8. Ábra: Severn-öböl alatti vasúti alagút

4. HAZAI ALAGÚTÉPÍTÉS KEZDETE

A hazai alagútépítés a Várhegyi alagút megépítésével kezdődött. A váralagút építésének igénye és gondolata már a Lánchíd építésével együtt vetődött fel. Ezt megalapozóan Gróf Széchenyi István 1845-ben társaságot alapított a budai vár alatti alagút létesítésére, amit azonban az 1848-as események meghiúsítottak. A szabadságharcot követően 1852-ben Ürményi József felelevenítette a gondolatot és létrehozta a részvénytársaságot az alagút létesítésére. A részvénytársaság az alagút tervezésével és a kivitelezés irányításával Clark Ádámot bízta meg. Az alagútlétesítés kiviteli munkálatait 1853. február 10-én kezdték meg és az alagutat a gyalogos forgalom számára 1856. március 6-án nyitották meg, míg a kocsi közlekedés 1857. április 30-án indult meg. Az építési költség közel félmillió pengőt tett ki.

Az alagút szerkezete egész hosszában téglaboltozat, a kapuzatok faragott kőből készült dór kialakításúak. Az alagút hossza 349,66 m, szélessége a kocsipálya szintjén 9,48 m. Az alagútkapuzat belső magassága a Duna felőli kapuzatnál 10,61 m, míg a Krisztinaváros felőli kapuzatnál 10,72 m. Az kocsipálya nyugat-keleti esése 1,8 %-os, a két kapuzat közötti szintkülönbség 6,10 m. Az elsősorban Budai Márgában elhelyezkedő alagút maximális takarása 39,20 m.

A Várhegyi alagutat követve 1861-ben megépült a Déli pu. és a Kelenföld pu. közötti vasúti szakaszon a kétvágányú Kis-Gellérthegy vasúti alagút, amely hossza 361 m. Az alagút patkószelvényű, magassága közel 6 m. Az alagút feletti kőzettakarás 4,35 - 19,14 m között változik. Az alagút Budai Márga, Fődolomit formációt harántol.

Az 1873 és 1911 között további 10 vasúti alagút épült meg 76 m – 780 m közötti hosszakban. Ezek közül kiemelendő a kis takarású Mőcsényi alagút a maga 780 m hosszával, amely szintén a Geresdi lösz-dombokban épült meg, ahogy 110 évvel később a tőle 20 km-re dél-keletre elhelyezkedő M6 autóút alagútjai.

Ezt követően a következő jelentősebb vasúti alagútépítés 1973 és 1979 évben volt az Abaliget I., illetve az Abaliget II. és III. megépítésével, továbbá a Nagyrákosi Balla-hegyi „Zuzsi” névre keresztelt alagút megvalósítása 2001-ben a magyar-szlovén vasútvonalon.

9. Ábra: Kis-Gellérthegy vasúti alagút

10. Ábra: Mőcsényi vasúti alagút

5. HAZAI KÖZÚTI ALAGÚTÉPÍTÉS

A jelenkori közúti alagútépítés megindításához a MAÚT 2003.-ban elkészítette és kiadta az ÚT 2-1.405 (e-UT 03.07.31.) Közúti alagutak létesítésének általános feltételei című Útügyi Műszaki Előírást, amely alapján kerültek megtervezésre, illetve kivitelezésre az ország első autópálya alagútjai az M6 autópálya Bátaszék-Vémend közötti szakaszán. Az alagutak hosszai 1331 m, 399 m, 865 m és 418 m, átlagos takarásuk 14-30 m, maximális takarásuk 40 m. Az alagutak lösz-dombokat harántolnak, fejtési keresztmetszetük kb. 100 m^2.

Az elmúlt években az M0-északi szektor alagútjai – „A" jelű alagút 2030 m, „B" jelű alagút 3345 m hosszal –, valamint az M85 Sopront elkerülő 780 m hosszú alagútjainak engedélyezési tervei készültek el és kapták meg az építési engedélyt.

A közeljövőben az M0-nyugati szektorán, az M100 autópálya M1-Esztergom, az M10 autópálya M0-Kesztölc és az M2 autópálya Vác-országhatár szakaszain létesülhetnek rövidebb, hosszabb közúti alagutak.

5. NEMZETKÖZI KITEKINTÉS

Magyarország domborzati viszonyait szem előtt tartva a nemzetközi kitekintést csak a kistakarású síkvidéki és dombvidéki alagutakra terjesztjük ki. A sort egy igazán síkvidéki ország alagút példáival kezdve elmondható, hogy a hollandiai vasúthálózaton 8 vasúti alagút, illetve a közúthálózaton mintegy kéttucat közúti alagút létesült, amelyek elsősorban folyók és tengeröblök, valamint repülőtér alatt vezetnek át.

További számos kistakarású alagút példák találhatók a szomszédos Ausztriában, Horvátországban, Szlovéniában, Szlovákiában, de távolabb Németországban, Spanyolországban, Olaszországban és világ többi fejlet és kevésbé fejlett országaiban is. Alagutak létesülnek ott, ahol azt gazdasági, környezetvédelmi, vagy akár zajvédelmi okok indokolják azt. És nemcsak közúti és vasúti, hanem a vízi közlekedést kiszolgáló vízi alagutak is épülnek.

11. Ábra: Hajózható alagút, UK Standedge-Marsden

Ausztriában, Németországban meglévő gyorsforgalmi útszakaszokat fednek be utólag környezetvédelmi és zajterhelési hatások csökkentése érdekében. Erre mutat példát az osztrák A10 autópálya Zederhaus települést elkerülő szakasza és a német Köln városán átvezető autópálya egy szakaszának lefedése.

12. Ábra: Zederhaus, A10 autópálya utólagos lefedése alagúttal

13. Ábra: Zederhaus (A), A10 autópálya utólagos lefedése alagúttal

14. Ábra: Köln (D), A1 autópálya utólagos lefedése

15. Ábra: Neumarkt (A), S10 gyorsforgalmú út befedése

A jelenleg érvényben lévő Útügyi Műszaki Előírás az elvakítás elleni védekezésként szilárd szerkezetű műtárgy építését írja elő. A közeljövőben kiadásra kerülő korszerűsített változatban ez az előírás valamelyest enyhül, de a probléma tényleges kezelése nagy forgalom és kedvezőtlen tájolás esetén továbbra is komoly szerkezeti megoldásokat követelhet meg. A nemzetközi előírásokban kevésbé található erre vonatkozó előírás, de a gyakorlatban találhatók építmények, amik e célból épülhettek. Ilyen az olaszországi Lecco városán (Comói-tó kelti partjánál) átvezető SS36 gyorsforgalmú út kelti portálja, valamint a madridi autópálya egyik portálja.

16. Ábra: Lecco (I) SS36 gyorsforgalmi út keleti portálja

17. Ábra: Madridi autópálya kijárat benapozás elleni védelme

KONKLÚZIÓ

Az elmúlt időszakban végzett alagúttervezések tapasztalatai alapján megállapítható, hogy az előttünk álló alagutak tervezése során különös figyelmet kell szentelni az alagutakat magába foglaló közutak tervezésére, úgymint annak vízszintes és magasság nyomvonalvezetésére, alagút portálok elhelyezésére, tájolására, forgalomtechnikai kialakítására. Ezekre már az első tanulmányi nyomvonalkeresés során tekintettel kell lenni.

Alagutak létesítésének szükségességét első sorban nem a magas hegyek jelenléte indokolja, hanem a domborzat változatossága, meredek szintváltásai – ami nem csak alpesi hegyvonulatoknál fordulhat elő -, továbbá a közúti és vasúti nyomvonal védendő környezeten, termőterületen, beépített környezeten való átvezetése. Alagút építése válhat szükségessé környezet- és zajvédelmi követelmények teljesítése, gazdaságossági megfontolások miatt is.

A külföldi példákban bemutat zajterhelés és elvakítás elleni védelemként épített megoldások és szerkezetek jó példával szolgálhatnak a hazai közúthálózat fejlesztése során. Úgy vélem van köztük meggondolásra ajánlható megoldás, ami hazai körülmények és adottságok között eredményesen és gazdaságosan hasznosítható lenne.

CONSTRUCTION OF THE STEEL STRUCTURES OF THE NEW DANUBE BRIDGE IN KOMÁROM-KOMARNO

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SUMMARY

The new Danube bridge in Komárom-Komarno is a cable-stayed bridge structure with one pylon, which is a Slovak-Hungarian joint investment with EU support. This bridge will be the largest console build bridge in Slovakia and Hungary. The bridge's pylon has very special shape, texture and material, which suspends the orthotropic deck. This bridge building is a real V4 collaboration, as the bridge will connect Hungary and Slovakia and the individual structural elements are made in the V4 countries, and the on-site assembly is carried out by the companies from these countries. Building this bridge is a very special task because of the very tight production and installation deadlines that require special solutions that make this project even more interesting.

1. INTRODUCTION

The new Danube Bridge will be the first one which meets the actual requirements and prescriptions for the load carrying capacity and the deck width on the common Hungarian-Slovakian Danube section; this way it will be able to serve the industrial development of the whole region on both sides of the river as well.

2. GENERAL SPECIFICATIONS

2.1. Bridge structure

In terms of location, the bridge connects Hungary and Slovakia. The Komárom-Komarno Danube Bridge will be the fourth largest-span bridge in Hungary. The bridge consists of five spans, three of them are located above the riverbed and the other two are on the Slovakian floodplain. The central main spans are bridged by a cable-stayed bridge structure with one pylon, which is joined by one bank span on each side as continuous beams. The main span, which is navigable, is 252 meters long, and the back side is split into two parts with an anchoring pillar.

The pylon has a 118-meter high asymmetrical layout, it is a steel pylon standing on the upstream side of the bridge, leaning transversally over the bridge.

The cable arrangement, bound into the upper part of the pylon is slightly divergent, has a so-called fan system, hangs the open deck girder in two cable plains.

The deck girder is a 600-meter long orthotropic steel deck with two main-girder open cross section beams. Cable connections are sectioned out in every 24^{th} meter of the main girder line.

The deck girder is assembled in three stages. On the Slovakian side with 250 meters interval moving, and on the Hungarian side with 80 meters interval moving, and between them there are 370 meters, which will be completed with the free cantilever method.

Fig. 1: Komárom-Komarno Danube bridge visualization

The customer of the bridge is NIF Zrt. and SSC, and after its completion, it will become the undivided joint ownership of Hungary and the Slovak Republic.

In the territory of Hungary, the Designer is the Pont-Terv Zrt. from Budapest, in the Slovak Republic it is the Dopravoproject a.s. from Bratislava. The production plans of the pylon were made by Ocelové Konstrukce Fasády from Brno.

75% of producing the deck girder was carried out at the Banimex factory in Bedzin, Poland, and the other 25% was carried out at the Komárom-Komarno shipyard, then the cross sections were pre-assembled at the Csepel port in Budapest. The railings on the bridge, the bridge investigator walkways, the lighting columns and other bridge secondary elements were made in Szigetszentmiklós near Budapest. The mounting of the deck girder is carried out by a Hungarian company.

The pylon-related structures were made in the Czech Republic, the bending of the pylon elements was made in Chrudim, the producing was carried out in Slany, the spiral staircase in the pylon was made in Skutec. The mounting of the pylon is being carried out together by a Hungarian, a Czech and a Slovak mounting company.

The stayed cables are made in Hlohovec, Slovakia.

The expansion joints are being produced in the town of Brandýs nad Labem-Stará Boleslav in the Czech Republic.

Fig. 2: Komárom-Komarno Danube bridge construction in May of 2019

2.2. Deck building technology

2.2.1. On-site installation and interval moving of Komárom-Komarno Danube Bridge on the Slovakian side

The deck girder cross-sections on the average of about 120 tonnes arrives at the work area on waterways on barges. Taking advantage of the on-site facilities, the cross-sections arriving on the waterway are lifted by a floating crane to the mounting support system which was built in the riverbed. After continuous welding of the cross-sectional suiting, the structure was launched incrementally towards Slovakia. Supporting the structure while launching is provided by the temporary mounting support system. The mounting support systems were not connected to each other, the deck girder between the mounting support systems arrives at the next mounting support systems with cantilever overhang. The sliding hydraulics were installed on the mounting support systems in the riverbed. During launching, the launching device was not fixed to the launching track and was relocated continuously after launching at each cross girder, but it was fixed to the mounting support system in the bed. The launching was made by two hydraulic cylinders with 100 tonnes capacity each operated from one hydraulic pump for launching. One cross girder advance required two-stage operation of the hydraulic cylinder.

To start the launching, 250-300-bar pressure set by the Designer was required, but to keep it moving a low, 200 bars were enough.

With the lifting hydraulics the sliding carriage is relieved, the carriage with a crossbeam is restored with the sliding hydraulics. The retracting of the carriages was carried out by retracting the sliding hydraulics, the other carriages with a 1 ton hoist. After the carriages were re-adjusted, the press could be loaded from the hoist onto the carriages. The above operations were repeated until the completion of each phase according to the phase plan. After reaching the position capable for receiving the new unit the track level of the bridge is set to the level corresponding to the level table. The fixed bearing design must be done. A complete process of launching, lifting, suiting, welding takes a week. After the launching we lift the bridge to the final bearing.

Fig. 3: 12th launching phase

2.2.2. On-site installation and interval moving of Komárom-Komarno Danube Bridge on the Hungarian side

On the Hungarian side, a 66-meter section between the bridgehead and the water pillar will be connected by a pair of launching beam, which will be supported in the middle by a mounting support system in the riverbed.

The floating crane lifts the deck girder cross sections to the launching beam, which are suiting and welding over a water pillar in a constant cross section. After welding a deck girder cross section, we pull it towards Hungary with the wheeled trucks and with a dragging hoist. Because the wheeled carriages have very low rolling resistance, there is no need to exert great power with the dragging hoist. A complete process of pushing, lifting, fitting, welding is expected to take one week in this case as well. After the launching the bridge should be lift to the final bearing.

2.2.3. Free assembly

Finally, will be built the deck between pile 2 and 3. This is a free assembly technology, where a shear connection was welded to the bridge elements. The function of the shear connection is to transfer the inserted structural element to the bridge console over the main holder, on the deck. The first step of the free assembly is installing the welding platform, lifting up the new deck element, geodetically adjusting and welding. After that we have to measure the shape of the console and start again the cycle.


Fig. 4: Construction of the pylon and the deck of the Hungarian side

2.3. Construction of the pylon

There is a total of 1 year to mount the 118-meter-high pylon, so the assembly and welding technology had to be chosen to meet this requirement. The construction of the pylon consists of 3 main units: The lower part with concrete section, the level from the track to the stay cables and the top part with the cable connections. Clark Ádám floating crane was used as a main machine for the installation of the lower, concreted part. One of the greatest advantages of the floating crane is that it has a load capacity of 200 tons and a good manoeuvrability. While piling and pile beam were built, they were transported on the road to the site, the elements of the lower, concreted part and, depending on their weight, were welded on pairs on floats. When the pile beam was finished, elements were placed in units of 150-170 tons, 10 meters long, 4 meters wide and 9 meters high and there were fitted. The geodetic setup was a major challenge for mounting in a constantly moving water installation area. As a result of doubles and triplexes, only every third link had to be made in the critical activities. To match the cross-sections of the lower concreted triangle, a facade scaffold system was built parallel to the pylon, which received a tread design against the drafts around the joints. One of the most interesting and challenging tasks of the project was the placement of the inclined beam. This beam is a 26-meter, 4-meter-wide, 2-meter-high, 80-mm-thick “H” -beam steel beam with a weight of 135 tons with 30 tons of concrete reinforced. It was pre-assembled in a lying position, then lifted onto the pre-assembled ironwork, fixed to it and the Clark Adam Crane set this 165-tonne structure from a horizontal position to a final angle of approximately 38°. Because there was very little space for the beam to be placed, the picking angle had to be set very precisely. This was achieved by lifting with 2 ropes of 10 meters and 2 ropes of 20 meters, and a hydraulic cylinder with a 300 mm adjustment capacity which was placed in the 20-meter rope. The crane began to raise above the 10-meter rope and then turned to the

20-meter rope at the point of lifting. After the picking up, the lower triangle was finished, the assembly of pillar concrete reinforcement was started.

The second stage is assembled with a three-level self-moving platform. The platform was designed and constructed in such a way that the continuous geometric change of the pylon - continuous reduction of cross-section, then growth, change of inclination angle - and avoiding the cable tie pipes, and then the installation of the slanting cables would not be a problem. The mounting platform has three levels. At the top level the elements are received and set, the second level is for cross-sections welding and the third level for corrosion protection and creeping. The scaffold was wrapped around with a fire-proof windshield. The creep is possible with the hydraulics attached to the ornamental flat elements on the side of the pylon. At the beginning of this mounting stage, the Clark Ádám crane raised the even elements until their lifting height allowed it, and then the 50-tonne tower crane lifted the following elements. The lower anchor structure of the tower crane was placed in the pile beam. Tower crane has built itself up until it reaches the 71.7-meter hook height. The tower crane has been able to work up to the height until now, so it has to wait until the pylon reaches the height of 69 meters, then the tower crane was attached to the pylon at a height of 53.3 meters and the tower crane can build itself up to a height of 106.5 meters, which is the second stage of the tower crane. The third step is to wait until the pylon is built up to a height of 105 meters and the console is install at 88.15 meters, the lower 53.3 meters pitch must be dismantled and the tower crane can creep up to a height of 126.8 meters.

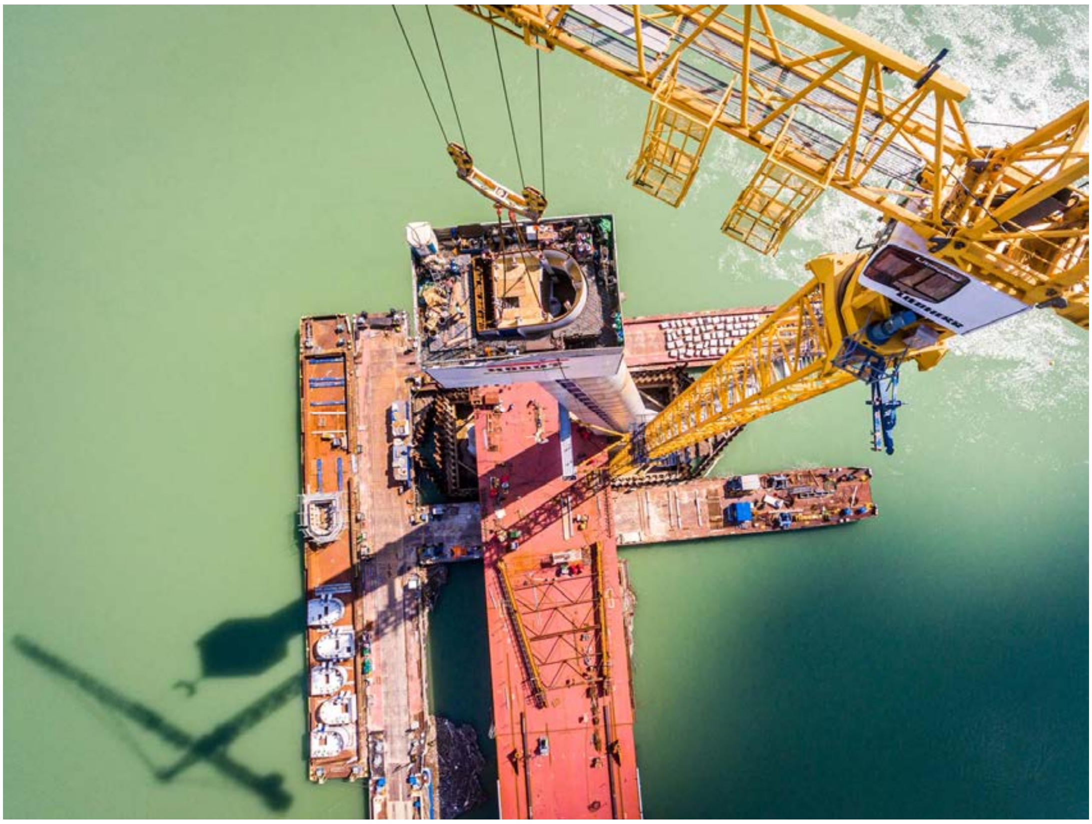

Fig. 5: Construction from the tower crane's perspective

In the second pylon assembly stage, the cycle of fitting a pylon element was started by lifting the pylon element, then geodetically adjusted, the elements clamped, and then welded around the mantle. In the case of such thick plates, the welding order is of immense importance, which is why the welding engineers have determined it by software simulation and experience. After the mantle was welded, the inner reinforcement ribs were welded. After the welding and their tests, the concreting was made in the pressed side. After welding and concreting, the pylons were re-measured geodetically and at the end of the cycle the mounting platform was crawled. One cycle took one week an average.

The third installation stage begins on the lower stay cable. The installation procedure is similar to the installation of the second section, but here, parallel to the welding work, the installation of the stay cables must be made. The biggest challenge in this installation phase is that the installation of the pylon at an altitude of more than 100 meters, the lacing and tensioning of the stay cables, and the mounting of the deck must go parallel and none of these activities can continue without the other. After installing the third stage, the mounting scaffolding and tower crane must be disassembled in the opposite order of their construction.

Fig. 6: Slovakian side of Danube Bridge in July of 2019

3. CONCLUSIONS

To sum up, the new Danube bridge building is really special for everyone who is working on this project. The construction of the new bridge will significantly reduce transit traffic in the inner part of settlements, so it will improve the situation and quality of life of the people there. This bridge is not only a modern, slender structure, but I believe it will be a symbol of Komárom and Komarno and a lot of people will talking about it in the future.

In my presentation I am going to speak about the details of the production and construction.

TOWARDS CONNECTED AND AUTOMATED DRIVING IN HUNGARY – THE CHANGING ROLE OF THE ROAD OPERATOR

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SUMMARY

Intelligent transport systems and services have been evolving significantly since last few years in Hungary. More and more emphasis has been given on introduction of new services and pilot applications, Hungarian Public Road Company plays an active role in this process. The field of C-ITS is one of the boosting areas and the topic of CCAD is also reflected. The road operators shall serve their customers and be prepared for autonomous cars, too, e.g. with road-markings and traffic signs. While there is a lot of uncertainty in the field of CCAD the HPR decided to launch a research. The paper intends to show the findings coming from this questionnaire and how HPR rely on the achievements of EU funded projects in order to cope with the changing role of road operators, supporting CCAD and also introducing future plans, and gives an overview of the C-ITS implementations carried out so far.

1. INTRODUCTION

The CROCODILE project is a co-operation between public authorities, road administrations and traffic information service providers as logical follow up of the previous projects CONNECT and Easyway. The CROCODILE corridor involves the Central and Eastern European (CEE) countries to ensure co-ordinated traffic management and control, resulting in high quality traveller information services on one of the most important road-corridors in an enlarged Europe. The CROCODILE corridor activities are focused on exchanging data and information between operators as well as countries to ensure harmonised cross-border traveller information services along the whole corridor. Even activities are carried out both, at national and/or European level; they are always seen in relation to an interoperable perspective serving the final goal of end user service provision. Main focus is laid on implementations with European dimension in order to ensure the efficiency in resource allocation using synergies and the European dimension assuring implementations at national level in a consistent way along the corridor. Several actors are involved in the activities, most of them directly related to road traffic and transport telematics applications. The main focus of the CROCODILE corridor was on free of charge end-user services including accurate traveller information including safety critical road services, traffic status information, and the information of available truck parking spaces. The basis for the service delivery to the end users are agreed Traffic Management Plans on cross-border level for areas along the corridor, where the need of increased safety and improved traffic flow is considered to be critical. The Hungarian beneficiary of the project is the Ministry of National Development, the domestic project partners are Hungarian Public Road Non-profit PLC (Magyar Közút Nonprofit Zrt.), and Budapest Public Road PLC (Budapest Közút Zrt.). In CROCODILE 1 (2013-2015) besides deployment of normal monitoring and traffic management tools, even new services were introduced, and pilot installations were extended, the main objectives of the work programme were:

- Establishing connections with neighbouring countries;
- Upgrading quality and availability of traffic data, automatic data exchange (DATEX II. node);
- Improving road safety, especially work zone safety (C-ITS pilot);
- Upgrading/extending services for road users: Traffic Information, Intelligent Truck Parking (Parking Management System).

Fig. 1: Crocodile corridor

In order to reach the goals, all data sources, and databases within Hungarian Public Road were 'opened up' to make data accessible for internal and external systems. Missing interfaces and data links were established in the frame of the system integration process. Now data received via the upgraded Datex Hub, after it is processed, can be visualised on the new graphic user interface of Traffic Control Software (FIR). Achievements of the integration can foster the improvement of traffic information services, and support traffic management activity, too.

While CROCODILE 1 focused on the Commission Delegated Regulation (EU) No 886/2013 of 15 May 2013 with regard to data and procedures for provision, where possible, of road safety-related minimum universal traffic information free of charge to users as well as Commission Delegated Regulation (EU) No 885/2013 of 15 May 2013 with regard to the provision of information services for safe and secure parking places for trucks and commercial vehicles both supplementing Directive 2010/40/EU of the European Parliament and of the Council, the ongoing CROCODILE 2 – as well as its' Hungarian twin project CROCODILE 2.0_HU – is focusing on implementing Delegated Regulation related to the provision of EU-wide real-time traffic information services which has been finalised already and should officially be published by the first half of 2015.

2. INTELLIGENT TRUCK PARKING

Heavy traffic is outstandingly high on motorway M1, Hungarian part of TEN-T Orient-East-Med core network corridor (Budapest-Vienna). This is the main transportation corridor between South-East Balkan countries, and West-Europe, and it is a main arterial of Central

Europe. Nevertheless parking capacities are very limited along this section the demand on parking lots cannot be satisfied, especially from dusk till dawn. At nights trucks are parking even on rest area roads, ramps.

The aim was to eliminate parking anomalies, to prevent the damages in the infrastructure, and to avoid the risk of accident, by providing the tool for planning the mandatory rest times for drivers. To reach our overall goal, an automatic parking information pilot system was Implemented in Easyway phase I. and II., which covered only 3 parking areas (110 parking lots). The system uses solely image processing to engage trucks in the parking area.

Fig. 2: Parking monitoring cameras, motorway M1 Óbarok (left) rest area (Photo: Zoltán Kapusi)

For building up a service, and to provide more options for truck drivers, in the frame of CROCODILE project, Hungarian Public Road Company deployed monitoring in service areas all along motorway M1 (from JCT of Bicske, to Austrian border) with higher parking capacity (all together 95 additional parking lots).

After covering the main part of motorway M1, the aim is to gather and prove availability information on the Hungarian part of the TEN-T core network, at least each 100 km (from motorways and expressways). So, on the missing eastern part of the Orient-East Med corridor (M0, M5 and M43 motorways) and on the Mediterranean corridor (M7, M0 and M3 motorways). In the framework of CROCODILE 2.0 project, altogether, 11 new service areas, at 6 locations are planned to be integrated (Fig. 3 planned: marked with blue, existing: marked with red).

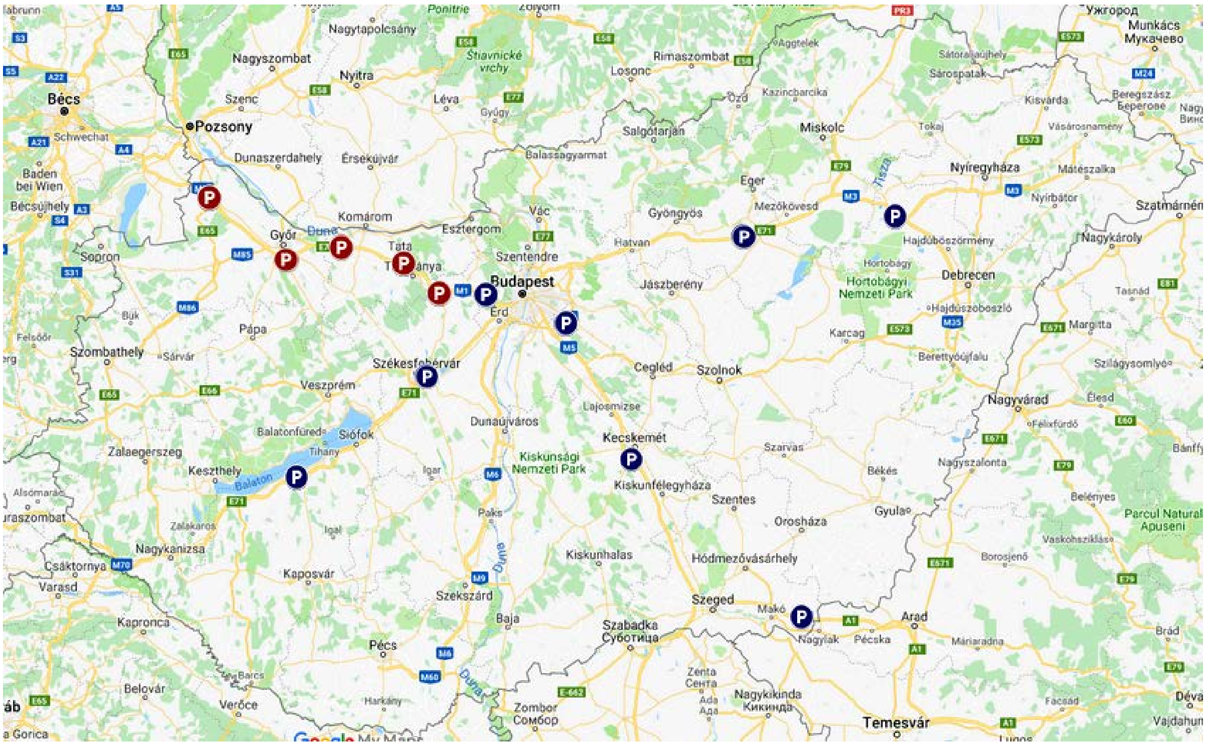


Fig. 3: Monitoring of parking lots by 2019

Tab. 1: Monitoring of parking lots by 2019

Motorway	**TEN-T corridor**	**Total number of parking lots (HGV)**	**Monitored parking lots by 2019**	**Ratio**
M0	Mediterranean	101	49	48.5 %
M1	Orient East-Med	316	224	70.9 %
M15	Orient East-Med	0	0	-
M3	Mediterranean	489	60	12.3 %
M43	Orient East-Med	94	60	63.8 %
M5	Orient East-Med	135	45	33.3 %
M7	Mediterranean	335	93	27.8 %
M70	Mediterranean	10	0	0.0 %
Total:		1,480	531	35.9 %

The possibility of showing cross-border parking information is also to be considered. Though the existing system in Austria is based on showing "Full" or "Free" message, and exact available parking place numbers in Hungary, it still might be possible to exchange the result from one system to the other, via the Datex Hub.

The real time parking information is available on roadside dynamic signs (VMS gantries and special information signs), as well as website, or in DATEX II. format via the Datex HUB of Hungarian Public Road and via the National Access Point of Hungary. In the border area (Austria, Slovakia, Hungary) the system is ready to integrate parking availability information from neighbouring countries, too.

3. INFRASTRUCUTRE TO VEHICLE COMMUNICATION PILOT

Cooperative ITS (C-ITS or cooperative systems), is a relatively new, emerging part of Intelligent Transport Systems, incorporates a branch of technologies and applications that allow effective data exchange through wireless communication technologies between components and actors of the transport system, very often between vehicles (vehicle-to-vehicle or V2V) or between vehicles and infrastructure (vehicle-to-infrastructure or V2I). More and more test sites appear in Europe to demonstrate the use of C-ITS, starting with pilot activities like the C-ITS-Corridor between Rotterdam-Frankfurt-Vienna, SCOOP@F in France, or NordicWay (including Denmark, Norway, Sweden, and Finland). Additionally, several smaller deployments on national scale (e.g. in Czech Republic) are carried out. The deployment of C-ITS is an evolutionary process that will start with the less complex use cases. These are referred to as "Day-1-services", encompassing messages about traffic jams, hazardous locations, road-works and slow or stationary vehicles, as well as weather information and speed advises to harmonise traffic. Using probe vehicle and infrastructure-related data, all C-ITS services shall be transmitted directly into the vehicles in a way that allows users to get informed but not distracted.

One of the main drivers to foster C-ITS deployment in Hungary was the involvement in the above mentioned European CROCODILE project. The project objectives were to improve the quality and availability of traffic data, to secure exchange of this data with neighbouring countries in DATEX II format, to improve road safety, i.e. in work zones, and to provide quality traffic information services to the drivers. In line with the above mentioned objectives, the Hungarian Public Road Company has selected part of its network for C-ITS services deployment, a 136km-long stretch of the M1 motorway between Austria and Budapest. The system is in operation from December 2015. The communication between RSUs and OBUs is thus far based solely on ITS G5. The system itself covered the following 'Day-1 services':

- Traffic jam ahead warning
- Hazardous location notification
- Road works warning
- Weather conditions
- In-vehicle signage
- In-vehicle speed limits

3.1. Central-side infrastructure

Central part of the I2V system was implemented as an embedded part of the upgraded Traffic Management System (FIR). Origin of the broadcasted information is either the so called Datex HUB, or alerts received from mobile units installed on maintenance vehicle. In the first case, Traffic Management System is only delivering information to the appropriate roadside units. Manual event data is recorded at the Dispatcher System (DIS), and available in DATEX II. format. C-ITS-S component is translating the DATEX II. XML file to DENM message, used in vehicular communication. The central unit is responsible for distribution of information: it transmits information from Traffic Management System via roadside units (R-ITS-S) to vehicles (V-ITS-S) ("downstream"), and gathers information from the maintenance fleet, and vehicles ("upstream"). Downstream information is received from the Traffic Management System, and after translation the messages are delivered to the appropriate RSUs (within a defined dissemination radius). The other direction is only partly implemented, yet. The system does not receive, and store probe vehicle data (so called CAM messages). Asides RSU status, upstream information is gained from the maintenance fleet.

3.2. Roadside infrastructure

R-ITS-S is a C-ITS system component located along the road. It is physically either fixed or temporarily setup or moving – e.g. at a road works safety trailer. It communicates to vehicles passing by. The R-ITS-S have the possibility to de- and encode C-ITS messages sent / to be sent over the ITS-G5 link. 27 fixed transceivers were deployed along the whole test section (13 emergency phone locations, 14 VMS gantries). Furthermore the rolling stock of maintenance centre Bicske was equipped with mobile units (road works warning trailers, heavy goods transport vehicles, crew vans, panel vans), too. A mobile R-ITS-S warning message can be created locally ("autonomous mode" or "stand-alone mode") or centrally in the C-ITS-S ("connected mode" or "augmented mode"). Stand-alone trailers / vehicles can obviously only distribute information that is derived from their own configuration – from the built in switches panel used to set the appropriate DENM message – plus the information obtained from a global navigation satellite system (GNSS) unit (time, position/direction, speed).

3.3. Vehicle component

The V-ITS-S component is demonstrating the operation of an on-board unit. Receives, and visualizes the information received from the roadside units over G5 link. The tool used for modelling a V-ITS-S is an Android mobile application.

4. FUTURE PLANS

Hungarian government placed strong emphasis on implementing and adapting new technologies in road transport. Hungarian Public Road Company is committed in fostering the spread of new technologies, and in using new communication channels to reach road users. Gearing up for the new challenges, to ensure the needs of connected and automated cars the road operator is participating in several projects, such as the RECAR (Research Centre for Autonomous Road Vehicles) responsible for the Hungarian test track, and C-Roads.

4.1. Participation in the harmonisation process (C-Roads project)

The C-Roads-Cooperation is regarded as an adequate instrument of Member States to continue their involvement in the dynamic field of C-ITS and to delve into more detail from a particular road operator perspective. Member States across Europe will install C-ITS pilot sites needed for the testing and later operation of "Day-1-Use-Cases" recommended by European Commission's "C-ITS platform". Therefore, Member States will invest in their infrastructure; OEMs and the industry will use that pilot test infrastructure to test components and services. It is in the task of several C-Roads-Working-Groups, organised in close cooperation with work performed in the Amsterdam Group, the InterCor project as well as in the EU-C-ITS-Platform and the EU ITS Platform, to elaborate specifications and agreements for harmonised and interoperable infrastructure based deployments. Based on these specifications and agreements technical and organisational issues will be piloted in several pilots across Europe, where different organisational frameworks, different technical approaches, different operating environments and different vehicle fleets will test and evaluate performance and feasibility of harmonised C-ITS systems and services.

Hereby harmonisation is always seen from the single driver perspective. Travellers using C-ITS services will not rely on specific communication technologies or service architectures. At the end drivers want to receive their relevant information through their respective human-machine-interface (in-vehicle or hand-held) using any communication channel. Aside the reliability, robustness, accuracy and in general quality of the received service, communication security and privacy are high important topics for end-users.

Even tested services as well as implemented systems will defer across C-Roads-Pilot-Sites, all installations will be done in a harmonised way by ensuring especially the mentioned interoperable end-user services based on international cooperation. To ensure this interoperability cross-site tests will be organised by the C-Roads-Platform and performed by the C-Roads-Consortium. All participating Pilot-Sites are committed in providing test-infrastructure both, road site as well as vehicle centred. The road site infrastructure will be open for other C- Roads-Operators and Partners to perform cross-site-tests. Consequently, C-ITS equipped vehicles will be used for cross-site-tests all across the C-Roads-Cooperation partners' networks. The stakeholders participating in the C-Roads-Cooperation see deployment of C-ITS as one of the most important tools to reduce congestion, get a safer and more secure traffic environment and to reduce emissions from traffic. C-ITS are in the phase to move from development to deployment, moving from research projects to real implementation. In this interface between research and deployment C-Roads will focus on piloting Day-1-C-ITS-services to show the maturity and to evaluate the impact of these services.

4.2. C-Roads Hungary

Core Members of the C-Roads platform are European States that agreed to work together to achieve deployments that enable interoperable and seamless cross-border C-ITS services for European travellers. At the current stage, sixteen European States committed to participate with their pilot sites. The C-Roads Platform remains open for other European States as well, as long as they are willing to actively participate. By the time the C-Roads Platform was set up in 2016, Hungary was just an associated member with limited possibilities, but as of November 2017, Hungary became one of the core members. From this time on, experts of Hungarian Public Road started their contributions to the specification activities of the working groups.

Besides C-Roads Hungary project, the parallel CROCODILE Phase 2 and 3 projects also aiming for new C-ITS investments on different parts of the Hungarian motor-/expressway network – but the most ambitious are of course the deployments within the C-Roads platform. Despite the fact that the C-Roads Hungary project started with a bit more than one year delay compared to CROCODILE Phase 2 Hungary, Hungarian Public Road Company managed to deploy its first "C-ROADS - Harmonised C-ITS Specifications for Europe – Release 1.3" compliant system carried out via CROCODILE Phase 2 project so called CROCODILE 2.0_HU. This deployment took place on the M0 ring-road around Budapest at 13 locations.

In Hungary, 2018 was the year of preparation for C-ROADS pilot deployment, closely following the specifications to be applied. Within this project, cellular communication is planned to be added besides ITS-G5 in order to support hybrid V2X, and additional Day-1 services is to be implemented such as Probe vehicle data, Green Light Optimal Speed Advisory / Time To Green and Signal Violation / Intersection Safety.

It also means, that besides the already considered one way I2V communication, we also plan to aggregate CAM data as a novel data source for traffic management in the V2I contexts.

Moreover, significant emphasis will be given on new services to the urban pilot planned in the city of Győr. This foreseen deployment has special circumstances – located close to the existing motorway pilot site, and also to Austria, and traffic is concentrated on kind of corridors mainly because of the main railway line is dividing the city into two parts. As a pilot, the most important goal is to gain experience from as great diversity of locations (eg. junction type, mounting, layout, etc.) as possible. We have identified 21 possible junctions to include into the test site in Győr, and the final locations for the implementation are under investigation. In addition, the coverage of M1 motorway between Győr and Hegyeshalom (Austrian border) will be improved, and the existing gap until the border of Budapest will be covered, too – just like the M7 motorway and M70 expressway as link to Slovenia. Within C-Roads, special attention is given to implement C-ITS services in light of cross-border harmonisation and interoperability, which are extremely important. Most likely, Road Works Warning – lane closure and Hazardous location notification – traffic jam ahead notification use-cases will be implemented cross-border as a first step, but the discussion among road operators are ongoing. As a basis for each kind of cooperation with partners regarding external or internal data exchange, the key element is the DATEX HUB of Hungarian Public Road Company. Within this system recorded event data from the dispatching subsystem as well as real time measured data from traffic management subsystems are also available – in DATEX II format, even for C-ITS purposes, in a highly integrated manner. The trilateral co-operation between Slovenia, Austria and Hungary (see the next section for details) will also rely on these investments.

Hungary has submitted a proposal for C-ROADS 2, as a continuation of the ongoing phase. The planned Hungarian work programme devotes particular attention to the creation of the urban test environment for the autonomous and connected vehicles in the town of Zalaegerszeg linked to the Automotive Proving Ground Zala (APZ), building on the experiences of the pilot project in the city of Győr. The idea is to focus on the CCAD contexts and to provide testing possibilities in open road conditions, too, which can even be re-simulated and analysed within the Test Zone later on.

4.3. Crocodile 3.

Traffic management, and cross-border co-operation is in the focus of the Hungarian work programme. Hungarian Public Road is willing to go on with work already started in the previous Crocodile 1 and 2 projects, the implementation of new, and upgrade of existing DATEX II nodes for the purpose of traffic management. Linked to this action the implementation of National Traffic Management Centre is planned, too. Traffic control is currently operating mostly in a decentralized way despite the fact that the criteria of centralization are given. Many of our systems support it such as Traffic Management System (FIR), Joint Traffic Light Management (JTR), Dispatcher system (DIS). The aim is to create a hierarchical system with two levels (regional and national). These levels correspond to the severity of an incident, too, but in principle regional centres would be responsible for the daily routine, and incidents with limited impacts. The national centre should handle events with regional, or cross-border effects. Main functions of the national centre:

- Supervision and coordination of regional centres
- Keeping contact with associated operators and public authorities
- Event management and network traffic management measures
- Executing cross-border traffic management measures

Besides international dimension, the work programme is focusing on Budapest area, TEN-T sections in the region, and urban-interurban interfaces. The aim is to support the traffic control of Budapest by the development of dynamic traffic management systems covering the whole network, based on the deliverables of CROCODILE 2 project on the surrounding network – especially on the M0 ring and the interfaces – in co-operation with the operator of the national road network for implementation of action plans defined in TMP on operative as well as on strategic level. The basis of the co-operation will be the development of DATEX II data exchange platform. Along with the development of the central software functions traffic monitoring equipment and communication network will be upgraded, too.

4.4. CCAD activities in Hungary

4.4.1. Zala Zone and the multi-level trilateral co-operation

An important part of C-ITS related investments in Hungary is connected to the Automated Proving Ground and Zala Zone, the area dedicated for real public road testing in Zalaegerszeg. In May 2016, the Government of Hungary decided on the implementation of the vehicle test track in Zalaegerszeg, with the vision to assist in strengthening of native automotive R&D capacities. In line with this, Automotive Proving Ground Zala Ltd. was established as the responsible company for managing and implementation of the test track: the task for this company is to elaborate the conception of the proving ground, to fulfil the investment, to build the related automotive and engineering knowledge, to establish competitive operation and to create the customer network.

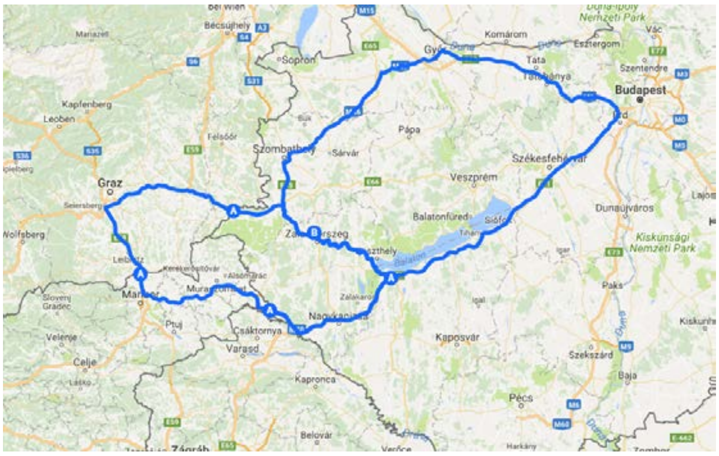


Fig. 4: Planned Routes for Automated Vehicle Testing within the trilateral co-operation concept

The Zalaegerszeg test track is unique, because it integrates all the traditional test track features focusing on driving and driving stability together with a complex research/development framework and advanced communication infrastructures to create a multi-level system for validating automotive solutions for the CCAD era. Therefore the proving ground provides not only dynamics tests for conventional vehicles, but it also allows validation tests for autonomous and electric vehicles. Hungarian Public Road was participating in the planning of the deployment phase of this test field. City of Zalaegerszeg is located close to Slovenia and Austria, and as C-ITS implementations are ongoing in both countries, the idea of setting up cross-border test loops has come up, connecting Zalaegerszeg, Maribor and Graz. Motivated by these opportunities the first trilateral conference was held in Zalaegerszeg, Hungary on the 28th of September, 2018, as the initial, official event of the manifesting trilateral co-operation. The co-operation is implemented on five different levels:

government/state (ministries), university, association (professional networks), business (test tracks), public road authorities.

4.4.2. Smart Road – M76

By setting up the test track, the demand raised to gain possibility for open road tests as well (with safety to be always the first concern). Besides the testing area of Zala Zone, the planned expressway M76 will be a part of the testing facility, too. This is the first road section in Hungary which is constructed with taking the requirements of automotive Tier 1s and OEMs into account already from the planning phase and aiming at CCAD test opportunities in real traffic conditions. The road itself will remain a public road and will be operated by Hungarian Public Road, with two main additional goals in mind: 1) to prove “ideal testing environment” for the automotive industry and 2) at the same time, up-to-date, well equipped “smart road” for everyone using the public road in order to gain the circumstances for a safer, smoother and more energy efficient way of transport. This effort could become easily the starting point for the improvement of the Hungarian road network. Eventful (diversified), strictly monitored and well-equipped roads with advanced (vehicular) info-communication technologies are the already identified key points for the ideal CCAD testing environment. Regarding “smart road”, the following options have been considered already: traffic monitoring solutions, V2X communication infrastructure, traffic management techniques, emergency phone system, use of renewable energy sources, and electric vehicle charging.

4.5. Questionnaire based survey on CCAD acceptance characteristics and adoption barriers

In order to discover user motivations, acceptance characteristics and potential adoption barriers of cooperative, connected and autonomous driving, and to support ongoing and future C-ITS infrastructure planning and deployment activities, Hungarian Public Road decided to initiate study by determining the methodology to be applied in 2018. As user acceptance behaviour and adoption decisions are often complex and mainly based on perceptions instead of facts, we decided to perform an extensive survey based on the efficient combination of questionnaires and expert interviews. The aim of the analysis is to assess public opinion on how traffic and the role of the road operator will change with the appearance of connected and autonomous vehicles. Communication between vehicles, infrastructure and other road users are also crucial to increase the safety of future automated vehicles and their full integration in the overall transport system. Cooperation, connectivity and automation are not only complementary technologies; they reinforce each other and over time, they are going to merge completely. We tried to introduce all the above aspects into our research to identify and prioritize the most important characteristics of potential users of future CCAD services.

The first phase of the data acquisition process was performed in April 2018 during the Transport Research Arena (TRA). The conference was covered areas like logistic solutions, digitalized planning and maintenance, dynamic transport management, digital infrastructure, such it offered a great opportunity to query and interview researchers, policy makers and industry representatives about their opinion. More than 100 people took part in our questioning process during the event (i.e., filled out the survey or participated for a short interview).

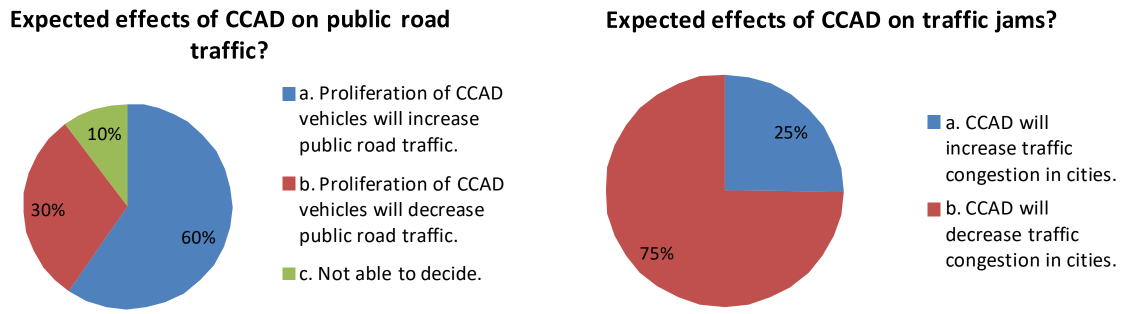


Fig. 5: Question block #1

In May 2018 we continued the data gathering within an online survey performed amongst the employees of Hungarian Public Road. The size of the database has been doubled reaching a total of 223 answers. The gathered dataset will be comprehensively analyzed and introduced in a separate paper to be published later on, also containing the results of a third phase of data acquisition to be performed within a wider scale online survey on a more heterogeneous set of interviewees to assure comprehensiveness. Here we highlight our preliminary results and initial charts.

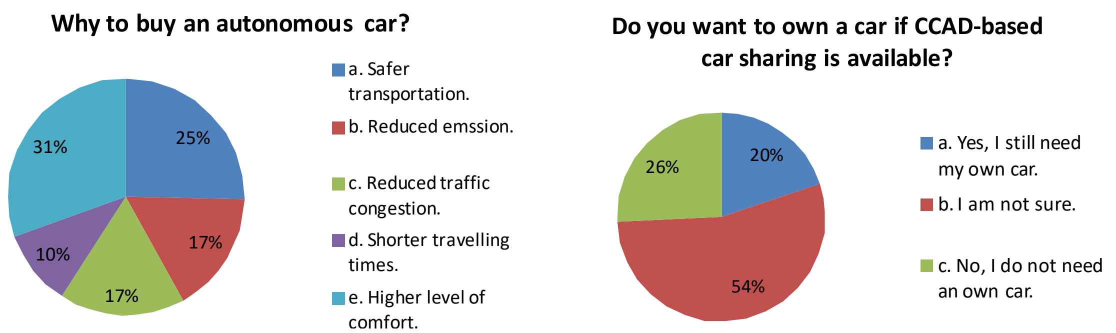


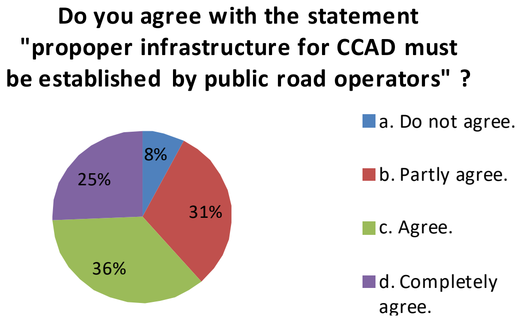
Fig. 6: Question block #2

A formal, structured answering process with predefined set of questions appearing in the same order was used to guarantee the reliability of the results. Two third of the respondents' work in the field of education/research or in the field of transport industry, almost the half come from the administration. 10% does not know how the autonomous vehicles are going to affect the traffic volume and only thirty percent thinks it will be reduced. A significant difference between the first and second phase of data collection lies here: the TRA participants had different opinion while the online responders expected mostly growing (Fig. 5, left). Answers in this block are interesting because the majority of respondents think that congestion will be reduced with the spreading of CCAD, only 25% expect increasing (Fig. 5, right).

A more comfortable and safer trip has come up with the two most anticipated benefits. Comfort gets a very high importance (31%) which can be explained by expecting the termination of driving with CCAD (Fig. 6, left). The question on car ownership in a CCAD era mainly gathered ambiguous answers with 54% of respondents not able to decide (Fig. 6, right).

The nearly two-thirds of respondents agree that it has to be the task of the road operator to design and create the proper infrastructure for highly automated and self-driving vehicles (Fig. 7, left). It is also interesting that that the road operator was marked for the task to handle the cyber security issues. Other answers show that the road operators are more trusted (77%)

than IT companies in privacy matters: if users must share their travel data (speed, route, etc.), they would rather give it to the road operator than software developers (Fig. 7, right).



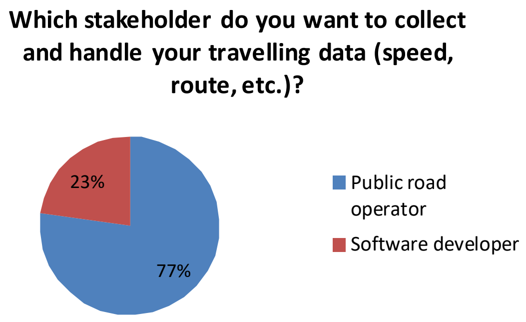
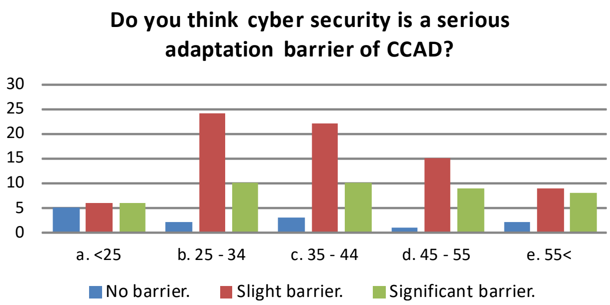


Fig. 7: Question block #3

The data management and protection can be a serious barrier against CCAD service proliferation. The huge majority of our respondents are afraid that their personal data will be misused. Only 8% of all sees lower risk in the issue, mostly the age group under 25 years (Fig. 8, left). The respondents were divided on that issue whose responsibilities are to ensure the cybersecurity. 44 percent of the participants thought so it is a road operator role (Fig. 8, right).



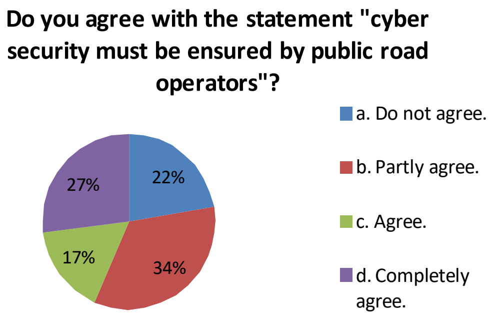


Fig. 8: Question block #4

5. CONCLUSIONS

ITS – Intelligent Transport Systems – are known as one of the most important tools to reach the environmental and sustainability goals in transport. ITSs are being applied to facilitate mobility ("making better use of existing infrastructure"), improve safety and help mitigating negative environmental impacts. ITSs can improve the performance of infrastructures better than building new infrastructure – i.e. quicker, less expensive and with lower or no environmental impacts. ITSs are developing together with technology and communication, and constantly pushing its limits. Hungary is trying to keep up with the progress and change. Although technological innovation causes plenty of challenges for road operators and there are a lot of uncertainties in the field of CCAD, Hungarian Public Road is focusing on supporting connected mobility. The ongoing deployments concerning a national C-ITS infrastructure and later a European C-ITS ecosystem are also helping this goal by providing more and more valuable information, best practices and harmonization guidelines. In order to facilitate the work-in-progress implementation efforts and to be well prepared for the new requirements of CCAD services and applications we will extend our analysis data with a third phase of data acquisition in 2019, which will be followed by an extensive analysis and comprehensive discussion of the integrated results.

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ROAD SAFETY INVESTIGATIONS IN DESIGN AND BUILDING

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SUMMARY

In November 2008, the Directive 2008/96/EC of the European Parliament and of the Council on Road Infrastructure Safety Management (European Parliament and Council, 2008) was published. The Directive includes definitions and principles on several road infrastructure safety procedures. The Directive applies to roads of the Trans-European Transport Network (TEN-T) in the member States of the European Union. To comply with this directive, the Hungarian parliament and the government made legal actions and technical guidelines were also developed. In the following chapters some results of the Hungarian implementation of RISM so far will be described.

1. SAFETY TRENDS IN HUNGARY

In 2018, around 25100 road fatalities were reported by the 28 EU Member States. This is a decrease of 21% compared to 2010 (14% in Hungary). It is a target of European Union to halve the number of road fatalities between 2010 and 2020. Achieving this goal seemed realistic a few years ago in Hungary, but it is now unachievable because the downward trend has turned into stagnation and slight growth (Fig. 1).

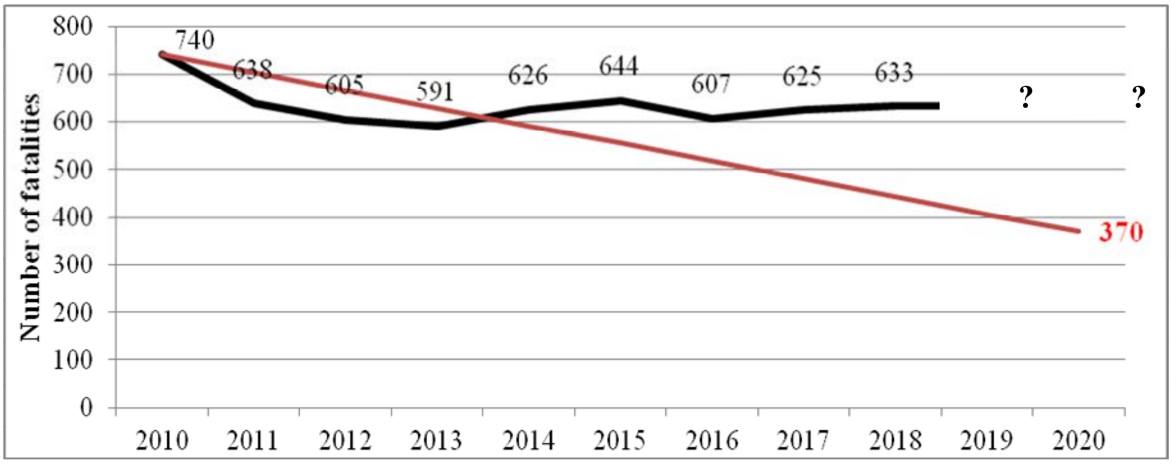

Fig. 1: Road fatalities in Hungary between 2010 and 2018
Source: Hungarian Central Statistical Office

2. LEGAL IMPLEMANTATION OF THE DIRECTIVE IN HUNGARY

The European Parliament and the Council of the European Union have adopted the directive on road infrastructure safety management (RISM) in 2008 (Directive 2008/96/EC). The goal of the directive is to ensure a high level of road safety on the roads within the European Union. The subject of the directive is the establishment and implementation of procedures relating to road safety impact assessments, road safety audits, the management of road network safety and safety inspections According to the directive, Member States were obliged to bring the necessary laws and regulations into force to comply with this directive by the end of 2010.

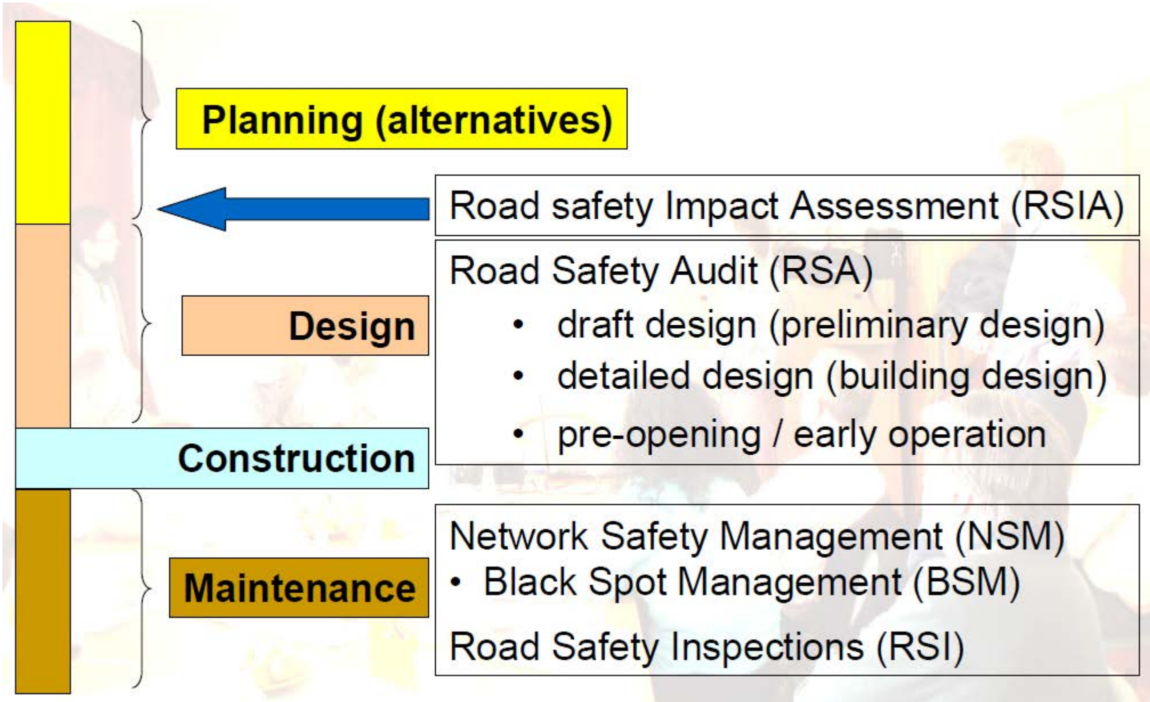

Fig. 2: RSIA, RSA, NSM, RSI – main procedures of road infrastructure safety management

In Hungary, an amendment of the Law on Road Transport was adopted by the Parliament in December 2010, containing regulations concerning the directive. Based on this amendment the Hungarian Government has adopted a decree on road infrastructure safety management in August 2011 with more details.

The European RISM directive applies to roads which are part of the trans-European road network, whether they are at the design stage, under construction or in operation. Member States may also apply the provisions of the directive, as a set of good practices, to national road transport infrastructure, not included in the trans-European road network. The Hungarian legal regulations are defining a wider scope, in phased implementation. From 2011, the regulations apply to the trans-European road network and to the motorways and expressways (these categories overlap each other considerably). From January 2014, the scope of the regulations was extended to all national main roads. A further extension came from January 2015 to all roads exceeding the traffic volume of 10 000 pcu./day, including municipal roads. Going eve further, several agencies require some elements of RISM (mostly Road Safety Audits) on lower level roads (e.g. cycle infrastructure projects). In the following chapters some results so far will be described.

To make infrastructure safer, the EU has also strengthened the rules on infrastructure safety management, the co-legislators having reached agreement in February 2019. In the future, infrastructure safety will be assessed more systematically and more proactively for more roads in the EU, helping to target investment. Transparency and follow-up will be improved, and the same advanced safety procedures will apply on roads linking major cities and regions as on the EU's strategic road network (TEN-T). It will also pave the way for automated assistance and prepare for autonomous driving across the EU, as well as the benefits these developments will deliver. Vulnerable road users will have to be taken into account systematically.

3. SOME RESULTS OF DIRECTIVE' PROCEDURES IN HUNGARY

During the application of the Directive, several road safety impact assessments, road safety audits, and safety inspections were carried out during road design, construction and operation. The recommendations of the auditors were not always accepted by the clients and the road operators for financial or scheduling reasons. The accepted proposals improved road safety, but the unapproved proposals were also useful for future planners and public road operators. In the following, I will present some of the auditor's comments on the roads design and the road network in operation.

The design and construction of bicycle facilities is primarily the responsibility of local governments, for which tenders are available in Hungary. One of the criterions for these applications is to conduct a road safety audit, which will greatly assist the work of local municipal officials, who usually have little road safety knowledge. I also present some of the bike lane auditor's suggestions.

3.1. Road safety audit of M85 motorway (draft design stage)

The Interim Temporary Junction Plan of M85 motorway an access passage between the two carriageways was planned to approach the roundabout. The difference in level between carriageways in the curve was a safety issue. At the auditor's suggestion, the other branch of the grade-separated junction was built in the first phase, eliminating the need for an operating passage (blue thick solid line).

Fig. 3: The design of the temporary junction and the constructed version

3.2. Road safety audit of M44 motorway (draft design stage)

Before the planned connection of the main road 44 to "Tiszaug", vehicles coming from the main road 44 from "Kecskemét" must move ~ 4 meters to the right, which can be ~ 40 meters long. The problem is the double curve of the main road was planned too short. Drivers coming from "Kecskemét" cannot be prepared for such a lane jump on a main road. The auditor's recommendation is to make the double arch longer and use a 30-meter continuous line before curves.

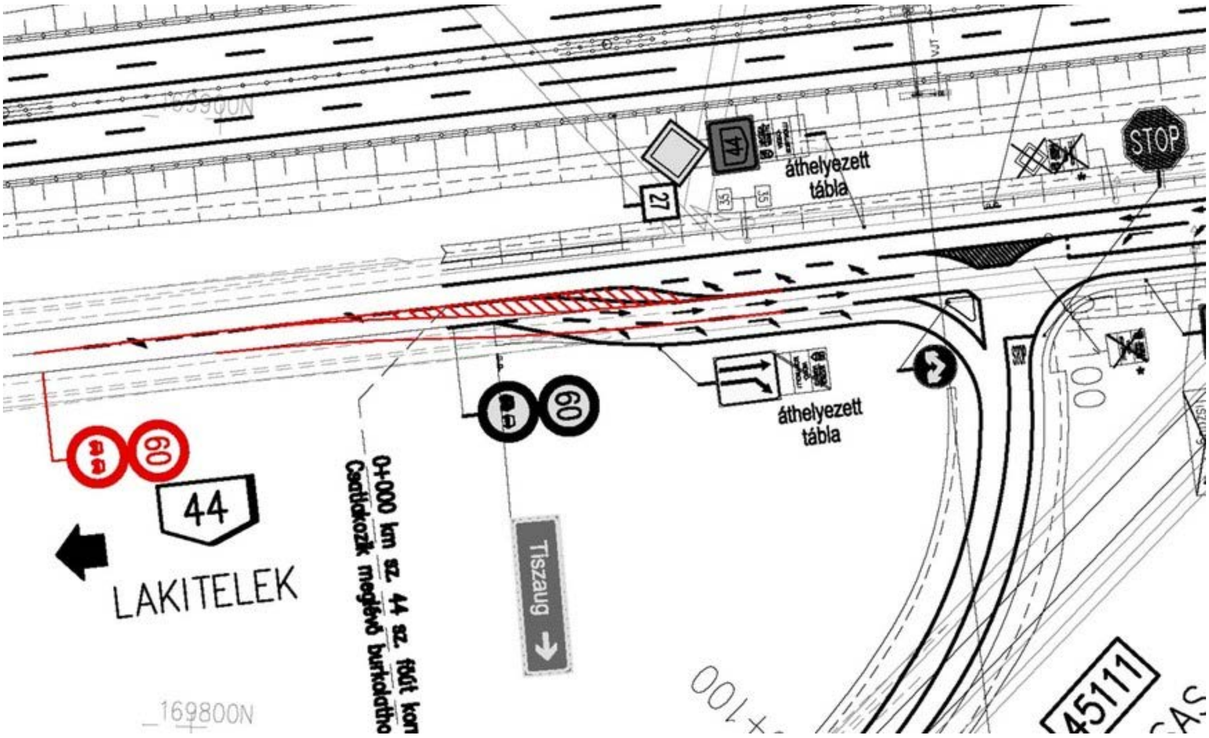

Fig. 4: The design of the junction and the recommendation of audit (red line)

3.3. Road safety audit of M85 motorway, between 80+775 – 89+980 kmsz. (early operation period)

In the location shown in Fig. 5, the 4 traffic lane structure change to dual carriageway separated by a steel guide rail. Problem: wrong-way entries carries an accident risk. This is especially the case in poor visibility conditions (backlight, fog). The auditor's recommendation are:

a.) "Do Not Enter" signs can be placed on a fluorescent background with a bypass sign (Fig. 5);

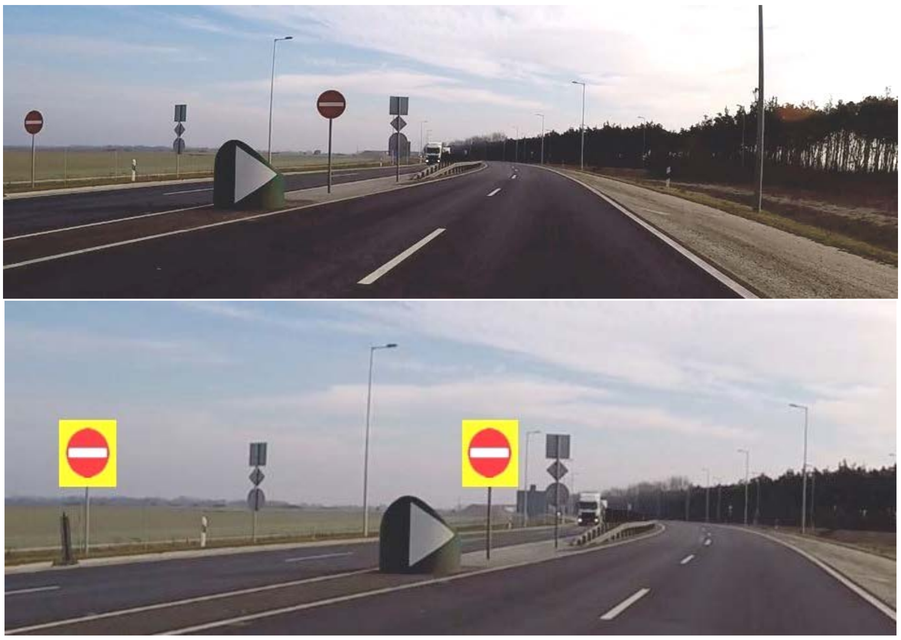
Fig. 5: The increased visibility of the sign

b.) painting the area closed to traffic in the separation zone;
c.) positioning of the collapsible, retroreflective delineators between a double closure line is approx. 50 m in length;
d.) Placing "Oncoming traffic" signs at a distance of ~ 200m from the end of the separation lane from “Csorna” direction.

3.4. Road safety audit of M85 motorway, between 13+800 – 16+100 kmsz. (early operation period)

On the connecting road under the M85 motorway, the signs meet and cover each other. Because of the covered signage, the signs cannot be interpreted, which can suddenly force drivers to brake. There is a risk of crashes. Recommendation: moving of signs for a better visibility of road signs, visual inspection of the final condition.

Fig. 6: Confusingly placed road signs

3.5. Road safety inspection of M19 motorway

The M19 motorway was built in 1977 to meet the technical and safety requirements of that time. The road safety review carried out in 2016 has identified a number of safety deficiencies, which is a large task, it will have to be resolved over several years. The largest problem is the unfavourable placement and lack of safety barriers (Fig. 7).

Fig. 7: Missing guard rails of M19 acceleration lanes

3.6. Road safety audit of cycle track in Tiszakécske (draft design stage)

Inside built-up area of Tiszakécske, a new bicycle infrastructure will be set up as a bicycle lane, on the road 4625 (Béke utca, Szolnoki utca) and a cycle track in the public space between the road surface and the land boundary. The Road Safety Audit Report contained 7 recommendations, two of which are outlined in the following figures.

The first problem is that the electric transformer station obstructs the visibility of cycle path (which continues as a cycle lane on the road), causing cyclists to unexpectedly appear (Fig. 8). The Auditor's recommendations:

a.) start the cycle lane in front of the electric transformer station;
b.) Applying a physical separator on the road surface at the start of the cycle lane (up to 1 meter in length) to provide protection for cyclists.

Fig. 8: The transformer station covers the cyclists

According to the plan, the trees will be preserved at the entrance of the fuel station. The second problem: cyclists will not be visible to drivers of vehicles entering the fuel station if the trees' foliage grows. Auditor's recommendation:

a.) trees shall be cut down;
b.) painting of the bicycle path crossing in red to clarify priority rules and smaller curve design of the right turning lane.

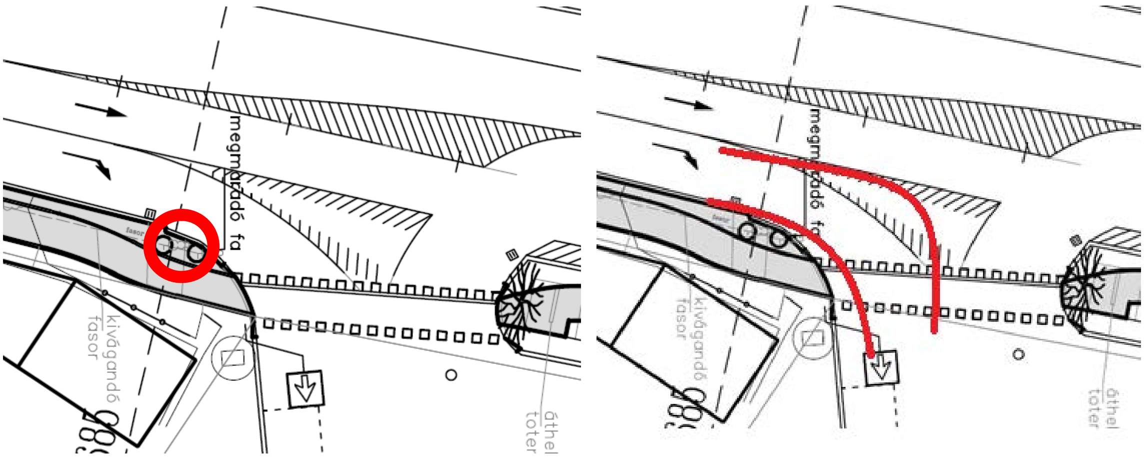

Fig. 9: Trees are covering cyclists

3.7. Road safety inspection of the cycle track in Szeged (Hero's square)

The bicycle lane passes through a parking space, but there is a street lighting pole in the bicycle lane (cost savings?). In order to protect the pillar or the cyclists (??), a guardrail was placed in front of the street lighting pole, which significantly reduces the space for the cyclists (in addition to that the trash is also stored here). The question is: who is defended by the railing: the pole or the cyclist? It would be sufficient to mark the column with a retroreflective road sign.

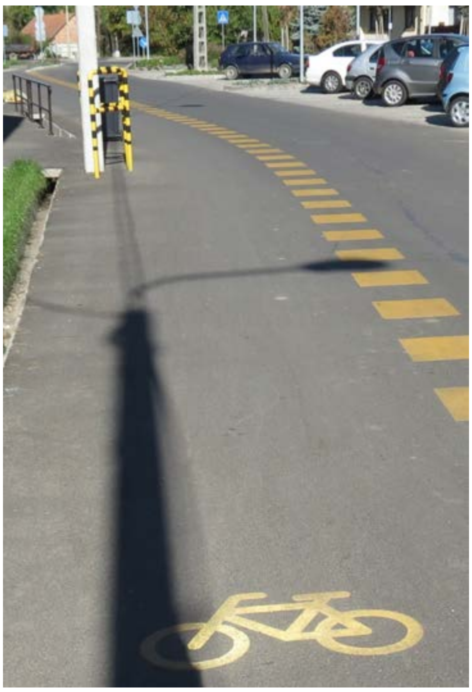
Fig. 10: Electric pole on the cycle path

4. CONCLUSIONS

Significant experience has been gained in the application of the RISM Directive by both auditors, designers and road operators in Hungary. Audit reports have been produced over the past decade containing valuable, useful suggestions, but unfortunately less successful materials have been produced too. After completing the audit courses, the views of designers and road operators have changed a lot towards safer solutions. The next task is to amend the domestic legislation and technical road regulations in accordance with the 2019 amendment of the Directive.

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SPECIAL PROBLEMS OF EMBEDDED RAILS IN URBAN TRACKS

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SUMMARY

Due to the increasing requirements for rail transport (noise and vibration load, decreasing duration costs), embedded rails are gaining more and more ground in our country. The practical planning of embedded rails is made difficult because of the fact that spring constants that are necessary for their planning are only available in a limited amount. Static, in some cases dynamic spring constants for test structures made of different embedding compounds are published by manufacturers, but the size of these test structures and their loading range are manufacturer specific. That is why every practical planning task requires that laboratory load measurements are made in advance, which is a time and cost-consuming procedure.

I developed a complex design method for embedded rails. On method I do not mean the designing of the concrete or steel structure, but the determination of the adequate embedding parameters (for example, the type of the embedding compound, the thickness of pouring, the rail profile) for the initial requirements (for example vehicle strain, expected support elasticity). FEM modelling is not needed in my method, it can be performed with an Excel program. At the same time, with an assembled structure it gives adequate results for different rail profiles, pouring thickness with the help of the former laboratory test results, so that the technical comparison of the different structures and the selection of the most adequate structure are made much faster and easier.

1. DILATATION OF THE EMBEDDED RAILS

The dilatational behaviour of embedded rails significantly differs from the ballasted and slab track with direct rail fastening. Fig. 1 shows an embedded rail. While the firstly flexible resistance is typical of the order of ~10 mm displacement, in the second and the third cases after a relatively small displacement (≤ 2 mm) the linear flexible period is followed by an ideally plastic period.

This latter behaviour can be modelled with softwares that enable non-linear calculation (providing spring stiffness and limit force), and in practical calculation the flexible period can be ignored and the plastic ballast resistance can be applied in well-known formulas. In the second graph there are force-displacement diagrams valid for ballasted tracks and slab track with direct rail fastening according to EN 1991-2.

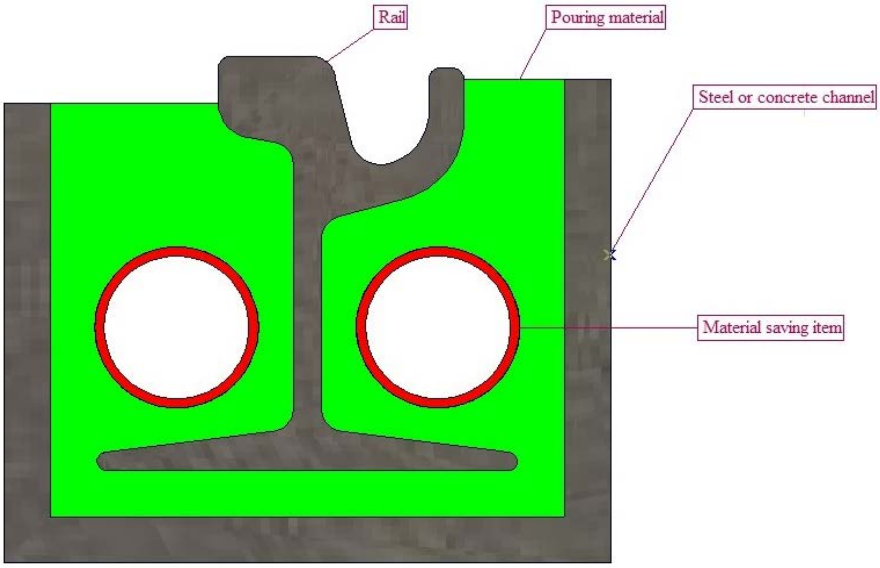
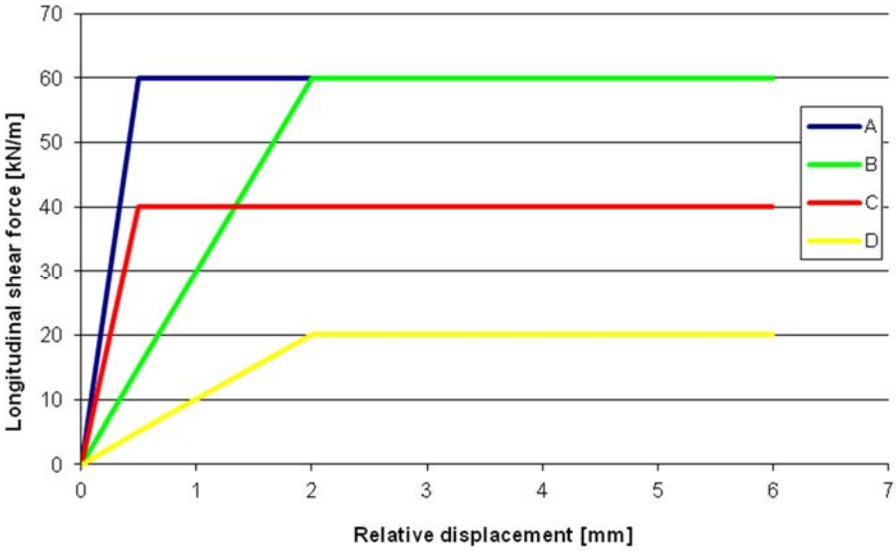


Fig. 1: Embedded rail structure



A Rail fastening on loaded track
B Ballast resistance on loaded track
C Rail fastening on unloaded track
D Ballast resistance on unloaded track

Fig. 2: Value of shear resistance in case of ballasted tracks and slab track with direct rail fastening (Figure above applies to 2 rails!)

The two different kinds of dilatational behaviour (flexible and plastic) show a significant difference when the graphs of dilatational forces and rail end movements are compared. I calculate the behaviour of a 60E1 rail in case of a 45 °C-temperature change. I applied 10 [kN/m/rail] value as plastic shear resistance, and I described the longitudinal stiffness of rail embedding with the value of 5000 [kN/m/m/rail]. When I established the length of the breathing section, I set 0.01 mm displacement as a limit condition and not complete stillness. The elasticity modulus of the rail is 206000 [N/mm^2], the value of the linear heat expansion factor is 1.2×10^{-5} [1/°C].

Figs. 3-6 below describe the changes of dilatational forces and the evolving displacement along the length of the rail.

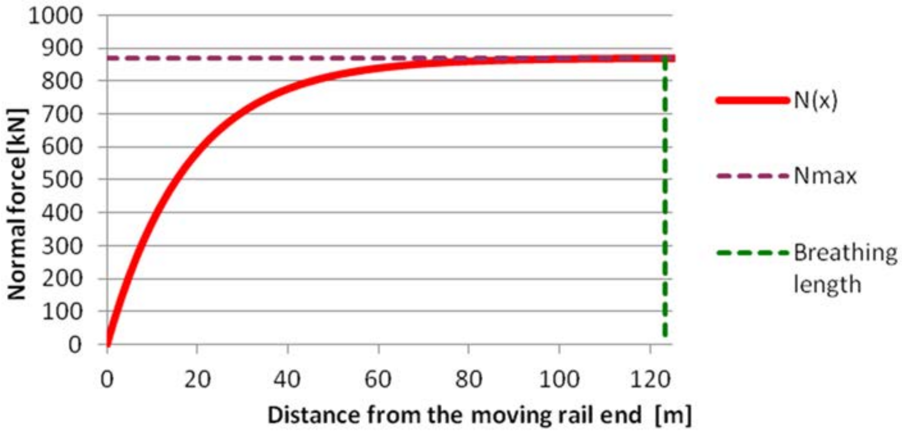


Fig. 3: The change of dilatational force along the length of the rail in case of flexible shear resistance (k=5000 kN/m/m, "breathing length"=123,500 m)

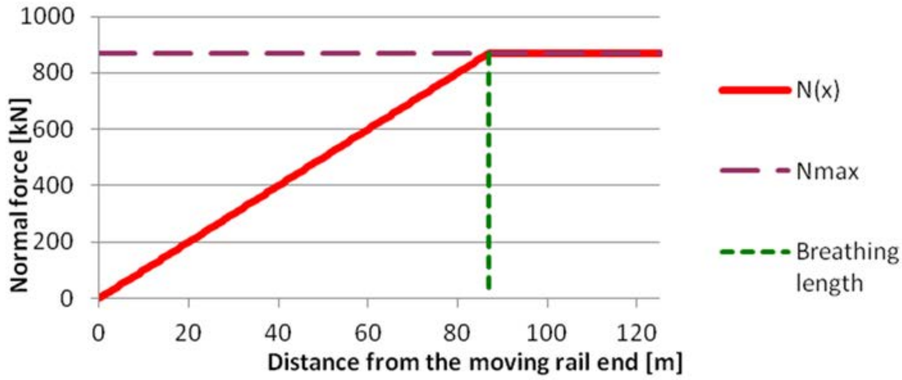


Fig. 4: The change of dilatational force along the length of the rail in case of plastic shear resistance (p=10 kN/m, "breathing length"=86,978 m)

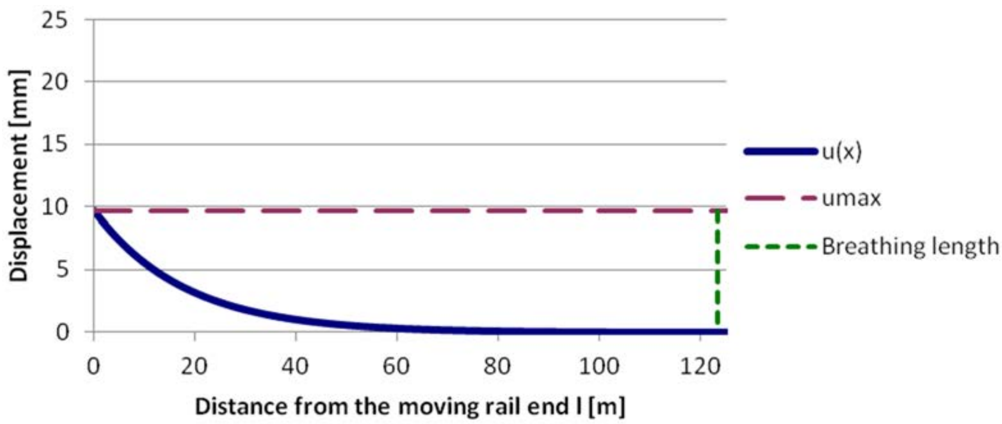


Fig. 5: The displacement of the rail along the length of the rail in case of flexible shear resistance (k=5000 kN/m/m, "breathing length"=123,500 m)

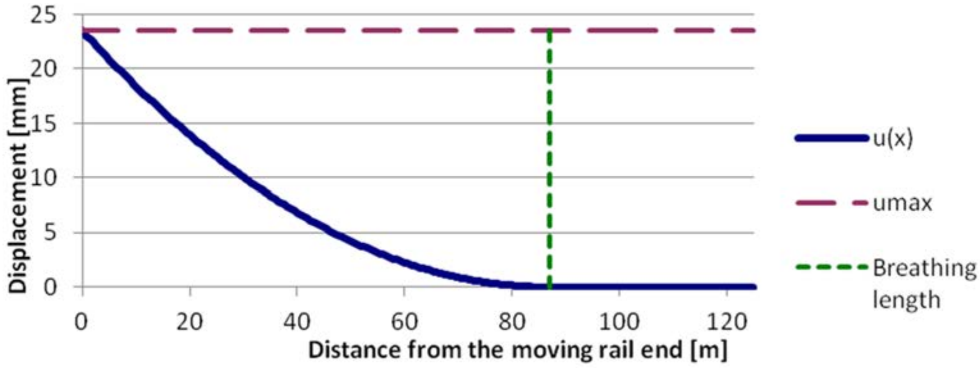


Fig. 6: The displacement of the rail along the length of the rail in case of plastic shear resistance (p=10 kN/m, "breathing length"=86,978 m)

Based on the figures above we can state that the hypothesis of the linear change of the dilatational force in case of embedded rails cannot be applied as in the cases of ballasted tracks and slab tracks with direct rail fastening. The evolving rail end movements are significantly smaller than the ones expected in ballasted track and slab track with direct rail fastening.

1.1. Analytical description of longitudinal behaviour

The linear shear resistance is described with the following formula:

$$\tau = k * u \tag{1}$$

where:
- τ: flexible shear/sticking resistance [kN/m],
- k: longitudinal spring constant [kN/m/m],
- u: displacement [m].

The differential equation describing the analysed problem can be given by the following formula:

$$\frac{d^2u}{dx^2} - \frac{k}{EA} * u = 0 \tag{2}$$

where:
- x: distance from the moving rail end [m];
- E: modulus of elasticity of the rail material [kN/m^2];
- A: cross sectional area of the rail [m^2].

After solving the differential equation we get the following equation for the displacement along the length of the rail (leaving out the deduction):

$$u(x) = \pm \frac{\alpha \Delta T}{\mu} * e^{-\mu x} \tag{3}$$

$$\mu = \sqrt{\frac{k}{EA}} \tag{4}$$

where:
- α: linear expansion coefficient of the rail material [1/°C],
- ΔT: temperature change [°C].

1.2. New formulas for practice

The analytical aspect of the dilatational behaviour of the embedded rail was described with the following interrelation:

$$u_{max} = u(0) = \pm\alpha\Delta T\sqrt{\frac{EA}{k}} \tag{5}$$

Using „c” system factor for each rail profile u_{max} can be rewritten into this very simple formula:

$$u_{max} = u(0) = \pm c\Delta T\sqrt{\frac{1}{k}} \tag{6}$$

where:

$$c = \sqrt{\alpha^2 * EA}\ . \tag{7}$$

The „c” system factor depends on the material features (α, E) of the rail and the cross sectional area in each rail profile (A). In this form the relationship is simpler and the need for calculation is much lower.

Tab. 1 shows values of „c” system factors for different rail profiles.

Tab. 1: Values of „c” system factors for urban tracks

Rail profile	c [$kN^{0,5}/°C$]
51R1	13.933
53R1	14.148
59R1	14.928
60R1	15.132
Ts52	14.182
35GPB	14.662
SA42-13	12.598

Note: During the calculation the modulus of elasticity of the rail material was E=206000 N/mm^2. The linear expansion coefficient was $\alpha=1.2\times10^{-5}$ 1/°C.

To determine the value of the complete moving length of the rail in the relation of the longitudinal spring constant is the next step. The breathing length considering the 45 °C rail temperature change can be given with the following power function:

$$z(k) = ak^b. \tag{8}$$

This general correlation derived from the examination of the results of the Excel calculations. The breathing length values were determined with different spring constants (k) and different rail profiles. Parameters „a” and „b” are summarized in Tab. 2 for urban tracks.

Tab. 2: Values of „a" and „b" factors of the power functions

Rail profile	a	b
51R1	15201	-0.5767
53R1	15323	-0.5757
59R1	16260	-0.5753
60R1	17039	-0.5787
Ts52	15725	-0.5781
35GPB	16053	-0.5762
SA42-13	13629	-0.5774

This applied formula provides the calculation of the breathing length in a closed form, for which there was no possibility before. Fig. 5 below describe the changes of the breathing length for urban tracks.

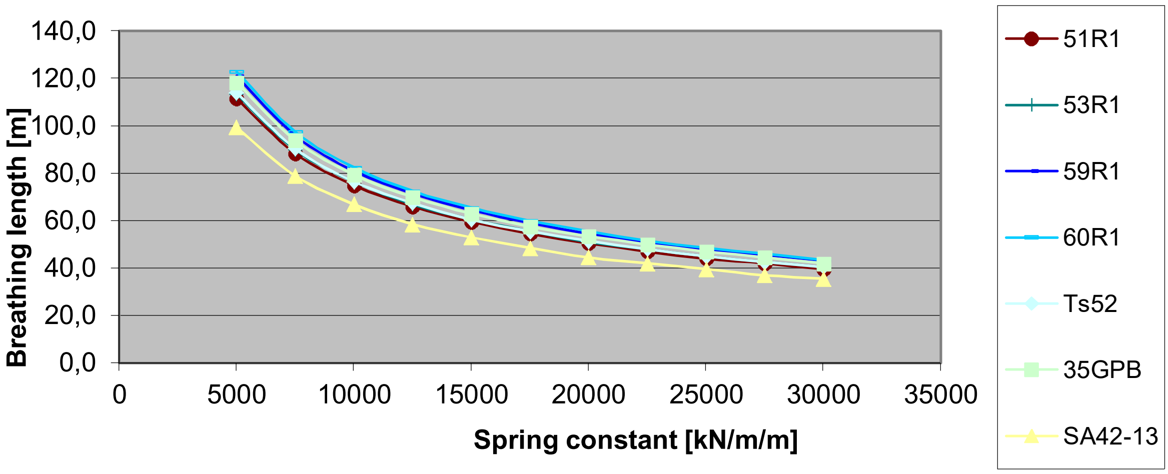


Fig. 7: Breathing length versus spring constant for urban tracks (ΔT=45 °C)

The (8) equation was modified with a relation depending on temperature change over 45 °C. The "t" factor makes a connection between the breathing length at 45 °C temperature change and values at higher temperature changes.

$$z(k, \Delta T) = ak^{b} * t(\Delta T). \tag{9}$$

In the formula above the only unknown factor is „t(ΔT)" and was defined for more rail profiles. Based on Excel calculations the following formula was determined:

$$t(\Delta T) = p * \Delta T + q. \tag{10}$$

The relations were determined for 45 °C < ΔT ≤ 65 °C rail temperature change. (The temperature at construction is +35 °C and the weld breakage at -30 °C, the temperature change is ΔT = 65 °C. Tab. 3 shows the „p" and „q" factor values for each rail profile for urban tracks.

Tab. 3: The values of „p” and „q” for urban tracks

Rail profile	p	q
51R1	0.0030	0.8670
53R1	0.0029	0.8724
59R1	0.0029	0.8715
60R1	0.0031	0.8608
Ts52	0.0029	0.8703
35GPB	0.0029	0.8704
SA42-13	0.0030	0.8680

The calculations were done for more rail profiles as earlier and about the same „t” values were received. So the following formula independent from rail profile can be suggested for practice:

$$t(\Delta T) \approx 0.0031 \cdot \Delta T + 0.8724. \quad (11)$$

I also dealt with the practical problem when the pouring of the rail happens over the upper limit of the neutral temperature zone. In this case there are two questions to answer: 1. what size is the opening gap in case of a possible winter rail or weld breakage, 2. if there is a damage in the adhesion of the embedding compound (e.g. tear from the channel wall) or in the embedding material itself (inner shear fracture).

Let's suppose that the allowed value of the opening gap in case of rail/weld rupture cannot be over 20 mm, this is corresponding to 10-10 mm rail end displacement. This can be considered as the permissible limit displacement. According to earlier studies the damage of the embedding compound does not occur at „u_{sup}” displacement, so it can be considered as the permissible limit displacement for the embedding material. Using the former correlations:

$$k_{min} = \Delta T^2 * f. \quad (12)$$

The „f” damage factor appearing in the formula above makes a connection between the rail profile and the maximum permissible rail end displacement. („c” is system factor.) The maximum permissible displacement is u_{sup} = 7.5 mm, there is no tear of embedding material from the rail or the channel wall, and the shear strength of the embedding compound does not exhaust. So a longitudinal spring constant lower limit value can be determined. If the spring constant is not lower than k_{min} value, the pouring material will be not damaged after a rail fraction in winter time, if the pouring was done at a higher temperature, above the upper limit of the neutral temperature zone. Tab. 4 shows the „f” factor values for each rail profile.

Tab. 4: The values of „f” factor in case of 54E1 and 60E1 rail systems

Rail profile	f [kN/°C^2/m^2]
51R1	3.4512
53R1	3.5585
59R1	3.9617
60R1	4.0707
Ts52	3.5756
35GPB	3.8218
SA42-13	2.8215

If the value of the spring factor is higher than the necessary k_{min}, the structure is resistant to the given temperature change besides the automatic fulfilment of traffic security requirements.

Note: Rail stresses must be checked in case of the fulfilment of $k_{min.}$

3. SUMMARY

In this article I presented the creation of a method that makes the calculation and the checking of embedded rails simpler and becomes closer to everyday engineering mentality. The other great advantage of the created method is that it does not require a FEM program, it is time-saving compared to it, because it needs no model-building. Using the introduced factors during my deduction I gave answers to practical problems such as the judgement of working out of the neutral temperature zone.

PAST AND PRESENT OF ROAD CONSTRUCTION TECHNOLOGIES

ÚTÉPÍTÉSI TECHNOLÓGIÁK MÚLTJA ÉS JELENE

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Hazánk az útépítési technológiák fejlődésében Európában az elsők között volt a XIX. század második felében és a XX. század első évtizedében. Az első három ország közé tartoztunk az aszfaltfelhasználásában. (Derna-Tataroson volt természetes lelőhely.) Az első világháború után elvesztettük ezt az országrészt, ezért áttértünk a beton alkalmazására. A két világháború között több mint ezer kilométer betonút épült. Az elvesztett második világháború után még rosszabb helyzetbe került az útszakma, mert sem bitumen, sem cement nem volt ipari méretekben. A szakmai kreativitásnak köszönhetően a gyakorlott mérnökök a legváltozatosabb eljárásokat és anyagokat próbáltak ki és alkalmazták. Nem szeretnek még a szakemberek sem visszaemlékezni a kátrány alkalmazására, pedig a negyvenes és ötvenes évtizedben hiánypótló kellősítő és kötőanyag volt, különösen az itatásos és kötőzúzalékos burkolatoknál. A második világháború utáni útkárok helyreállítását megelőzte a hidak, különösen a nagyobb nyílású folyóhidak helyreállítása. Az útkárok megszüntetése gyakorlatilag a folytonossági hiányok, kátyúk kijavítására irányult. A háború után megmaradt kevés korszerűtlen aszfaltkeverő géppel több évtizedig készült hidegaszfalt, melyet vasúti szállítással juttattak el az egész ország területére. A motorizáció lassú térhódítása a hatvanas évtized végére és a hetvenes évtizedre volt jellemző, amely egybeesett a téli és a tavaszi olvadási és fagykárok tömeges jelentkezésével. A hézagos, szórt útalapokon a kevés víztávoltartó kötőanyag és az aszfaltok hiányát pótolni kellett, ezért a költséges és sok élő és gépmunkát igénylő útkorszerűsítések helyett az aszfaltszőnyegezés lett a vezértechnológia. A nagytömegű meleg hengerelt aszfaltok építéséhez a világháború után alig maradt gépi felszerelés, ami volt, az is korszerűtlen és kis kapacitással rendelkezett, ezért elkezdődött az akkor korszerűnek minősíthető hazai C-25-ös, 25 tonna/óra teljesítményű keverőgépek gyártása. A hetvenes évtizedre az aszfaltszőnyeg készítése volt jellemző, nem ritkán évi 6-7 millió tonna aszfalt felhasználásával. (Ezeket a mennyiségeket azóta sem sikerült túlszárnyalni, sőt gyakran a felét, vagy annál kevesebbet sikerült csak teljesíteni az időnként jelentős autópálya-építés ellenére is.) Az ország gazdasági helyzetének romlása – a nyolcvanas évtizedre – azonnal éreztette hatását a költségvetésből finanszírozott útépítéseknél. Az állami, szakmai irányítás a gazdasági megszorításokra azonnal reagált, és olyan technológiaiválaszték-bővítéssel igyekezett úrrá lenni a helyzeten, hogy minél nagyobb útburkolat-felületen lehessen állagmegóvó technológiákat szorgalmazni. Előtérbe kerültek a vékonyabb aszfaltburkolatok, és a permetezéses felületi bevonatok. Bevezetésre kerültek a hat korszerű gyárban előállított bitumenemulzióval készült technológiák, elsősorban a felületi bevonatok. A hézagos, utótömörödő burkolatokat lezárva a vizek távoltartása volt a cél.

Évente 25–30 millió négyzetméteren készültek az alsórendű úthálózaton jó minőségű felületzárások. A főutakon az aszfaltvastagság csökkentésével, valamint a bitumen és az aszfalt modifikálásával sikerült tartós burkolatokat készíteni. A közúti szakmában mind a kivitelezők, mind a fenntartó szervezetek újabb és újabb technológiai újdonságokkal kísérleteztek mind a kötőanyagok, mind az adalékanyagok tekintetében. Az elégtelen teherbírású utakon jelentős deformációk alakultak ki, melyet az aszfaltozás előtt kiegyenlítő aszfalttal hoztak profilba. A néhol több cm vastagságú kiegyenlítő aszfalt a ráhelyezett egy vagy két réteggel együtt egyenetlenül tömörödött, ezért hullámos lett a burkolat attól függetlenül, hogy az aszfaltszőnyeg a készítéskor nem volt hullámos. Ezt küszöbölte ki a hideg burkolatmarással, a csiszolómarással előkészített felület, amellyel a kiegyenlítő aszfalt mennyiségét is csökkenteni lehetett. A modifikáló szerekkel az országhatáron kívülre is eljutó technológiai újdonságot alkalmazott szakma például az M7-es autópálya tönkrement betonburkolatának lefedésével, 3 cm vastag, modifikált bitumennel készített aszfalttal. (Egyes szakaszokon 15 évig is jól szolgálta az egyre növekvő forgalmat!)

Sikeresen működött az autópályák burkolataként az érdesített homokaszfalt (ÉHA), valamint biztató korai eredményeket lehetett tapasztalni a főúthálózaton alkalmazott ún. „vízáteresztő" drén vékonyaszfaltokkal is. A gyors tönkremenetel nem az aszfalt hibájából, hanem fenntartási hiányosságból, és a nem kellő helyen történő alkalmazásból adódott.

A korlátozott mennyiségben található, jó minőségű eruptív kőzet kiváltását sikeresen oldotta meg a szakma – megfelelő forgalmi kategóriákban – kvarc és mészkő adalékanyagokkal, valamint a kohósalak alkalmazásával. A különböző technológiai műhelyekben termékeny kísérletezés és a megfelelő eljárásoknak az adott forgalomnak még éppen alkalmas helyeken történő bevezetése általános volt az országban. Például itatásos, vagy permetezés eljárásokhoz hasonló új, könnyűaszfalt burkolatok készültek. Kezdeményezések voltak a hiányosan rendelkezésre álló kötő- és adalékanyagok kiváltására is. Az utak szélességének hiányait iparszerű tömeges szélesítéssel, kampányszerű kivitelezéssel igyekeztek a fenntartó szervezetek csökkenteni. Tört és osztályozott kohósalakkal, különböző szemszerkezetű zúzott kvarc anyagokkal, mechanikai stabilizációval épültek a kisebb forgalmú utak szélesítései. Az olyan területeken, ahol rendelkezésre állt a jó minőségű finom kvarc, ott cementstabilizáció volt a szélesítés anyaga. A cement kötőanyag korlátozott rendelkezésre állása miatt a széntüzelésű erőművekben keletkező erőművi pernye megfelelő helyettesítő stabilizációs kötőanyag volt a hetvenes és nyolcvanas években. Ezt az egészséges folyamatot lassította, majd teljesen megállította az 1990-es hazai politikai rendszerváltás. Az útépítő iparágat (sajnos a kő- és kavicsbányászatot is) érintő privatizáció az első időszakban új irányba terelte a technológiai váltásokat. Ugyan az első időszakban a drága, nagy technológiai célgépek megjelentek az útépítő szakmában, és sikeresen alkalmazták a legkorszerűbb takarékos eljárásokat (melegremix burkolatfelújítás, hidegremix burkolatalap-homogenizálás), azonban amint világossá vált, hogy ezek az eljárások az egyszerűbb és nagyobb profittartalmú melegaszfalt technológiának előnyös versenyeljárásai, gyorsan elmaradtak a piaci verseny kínálatából. Az aszfaltburkolatok felújításakor keletkezett felmart aszfalt újrahasznosítása a nemzetközi gyakorlathoz képest késve, de hazánkban is általánossá vált. A melegaszfalt-keverő telepek többnyire rendelkeznek olyan kiegészítő berendezésekkel, amelyekkel az adagolás megoldott. A teljes keletkezett felmart aszfaltmennyiséget ezzel a módszerrel nem tudja a szakma hasznosítani, ezért még előfordul, hogy kisebb értékkel hasznosul, pl. burkolatalaphoz vagy különböző kiegyenlítő rétegként, de még az útpadkába is alkalmazzák.

Adós a szakma a hidegen történő aszfaltkeverékben történő újrahasznosítással, pedig több évtizede jó tapasztalatok vannak ezzel az eljárással is. Nem kedvezett az igen takarékos és az értékelemzés szemléletében szükségszerűen alkalmazandó különböző remixeljárásoknak az egyre bonyolultabb és szinte teljes egészében a jogász szakma által művelt közbeszerzési gyakorlat sem. Ennek köszönhetően is volt olyan, több évig tartó időszak is, amikor sem a meleg-, sem a hidegremix eljárások nem készültek iparszerű mennyiségben. Hátráltatta a teljes körű alkalmazásukat a mindenkori erősen átpolitizált állami irányítás.

A különböző irányultságú kormányok más és más infrastruktúrák fejlesztését támogatták, így az autópályák építését, vagy a városi elkerülő szakaszok megvalósítását, vagy az aszfaltszőnyegek kivitelezését részesítették előnyben. Ezekben az esetekben háttérbe szorultak az értékelemzés szemléletében szükségszerűen alkalmazandó eljárások és technológiák. Ez a tendencia részben napjainkban is érzékelhető, például az alsórendű úthálózat leromlását lassító felületi bevonatok gyakorlatilag nem készülnek, miközben sok ezer kilométer leromlásnak indult, nyitott felületű burkolaton lehetne a gyors leromlást lassítani. A technológiai választékot még mindig szaporítja az öntöttaszfalt általános alkalmazása, „virágzása”. Az öntöttaszfalt kiváló tulajdonságai indokolják, hogy szűk körben, például hidak szigetelésénél, vagy különleges termek aszfaltozásához felhasználásra kerüljön. Sajnos hazánkban még sűrűn alkalmazzák a városokban járdák burkolására, vagy közművek meghibásodása utáni burkolat-helyreállításhoz. Különösen indokolatlan ezekben az esetekben, hiszen korszerűtlen gépekkel és nehéz fizikai munkával kerülnek beépítésre. A hagyományt nehezen hagyják el ezek a szakma peremén működő kis magánvállalkozások, de azért is, mert tömeges megrendelés között válogathatnak. A közúti mérnöktársadalom a történelem során miden nehéz gazdasági és társadalmi közegben igyekezett helytállni a mindenkor rendelkezésre álló szűkös források és anyagi technikai körülmények között. Jellemző volt az a tendencia egészen a XXI. század küszöbéig. Azóta a megnövekedett források elkényeztették a szakmát, eluralkodott a mennyiségi szemlélet, és a már bevált kevés technológiával valósulnak meg a jelenkori projektek. Az elmúlt évek tapasztalata azt mutatja, hogy a szűk technológiaválaszték ellenére a pusztán melegaszfaltból készült burkolatok minősége is hagy kívánnivalókat, az autópályákon a technológiaváltás eredményeként épült betonpályákéhoz hasonlóan.

MEGYERI BRIDGE: CABLE-STAYED BRIDGE ON THE MAIN DANUBE BRANCH

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SUMMARY

The Megyeri Bridge, the first cable-stayed bridge in Hungary, is located on the northern section of the M0 motorway bypassing Budapest. Construction began in 2006 and it was inaugurated on September 30, 2008. The entire Northern Danube bridge is 1862 m long, and it actually consists of 5 separate bridge structures. The subject of this article is the 591 m long cable-stayed bridge over the main Danube branch.

The bridge has three openings and support spans of 144+300+144 m. It has a two-pylon design; the pylons have an "A" shape and a height of ~100 m above the substructure. A total of 88 cables support the steel truss in a fan shape in two planes. The bridge has been delivered with a split lane of 2×2 lanes, but 2×3 lanes can be provided without rebuilding in the future.

1. GENERAL DESCRIPTION

The M0 Megyeri Danube Bridge with its length of 1862 meters is the longest river bridge in Hungary, consisting of the following five connected and different bridge structures. The general description of the whole bridge and its developing process has been published by Hunyadi (2008).

Tab. 1: Bridge structures

Name	Span in [m]
Flood-bridge on the left river (Pest) side	37.15 + 2 × 33.00 + 44.00
Main Danube branch bridge	**144.00 + 300.00 + 144.00**
Flood bridge on the Szentendre Island	41.00 + 10 × 47.00 + 46.50
Bridge over the Szentendre Danube branch	93.00 + 144.00 + 93.00
Flood bridge on the right river (Buda) side	41.43 + 3 × ~44.00 + 43.50

The 590 m long bridge on the main branch of the Danube is the first cable-stayed bridge in Hungary, which is a remarkable update to the existing bridges built over the Danube in terms of structure and construction.

It took nearly 14 years from the initial design phase to the start of construction, which included the following planning milestones:

1991-92	coordinating northern sector route plans
1993	study plans for the bridge
December 2001	licensing plans submitted
September 2004	final building permit issued, elaboration of tender plans
January 2006	construction plans started

This article presents the main load-bearing structural elements, the technological processes required for their construction and the construction phases. In the section about designing the structures, we will cover the effects of the construction, which had to be taken into consideration when preparing the construction plans.

Fig. 1: Megyeri Bridge over the Danube (Photo: Márta Hegedűs)

2. SUBSTRUCTURES

Based on the results of the excavation, a 1.50 m drilled pile foundation was made for the substructures. The piles were encapsulated in a thin, medium clay layer of heavy-duty oligocene gray marl.

2.1. Common pillar

The cable-stayed bridge and the connecting reinforced concrete flood bridges rest on the so-called common pillars No. 5 and 8. The substructures were placed at the edge of the riverbed, in the floodplain. They are identical in construction, each with 8 pieces of 19 m long piles with a diameter of 1.50 m arranged in two rows for both common pillars. The pile strap beam is 49.40 m × 7.50 m in overall size, 2.50 m in height.

The sides of the lower 5.50 m high part of the ascending wall were made with 1:20 inclination, their two ends are pointed arched and built with granite nose stone. In the subsequent part of the ascending wall, the walls perpendicular to the bridge axis are vertical, with the front/end of the wall inclined at the same angle as the railing. This section also includes the anchoring of the cable-stayed bridge, which consists of 2 cables with 16 strands per side. The anchor cables are designed as pendulum columns so that no other constraining force is created in the structure due to the effect of the bridge movements.

The structural beams and the bearing stools were integrated with the ascending wall. Underneath the box girder of the cable-stayed bridge, Maurer pot bearings moving in all directions are incorporated to convey vertical reactions. The transverse support of the structure is carried out by a special wind shoe built into the axis of the bridge.

2.2. Riverbed pillars

Pillars 6 and 7 of the bridge are the substructures of the bridge pylons, they are built in the riverbed using a reinforced concrete cofferdam stiffened with steel trusses. Due to the size of the substructure (70.00 m × 16.50 m), the crust elements used for enclosing the work space were built from 3 parts. The cofferdam elements were further covered with an additional 5.00 m high screw-mounted (thus removable) steel guard wall.

Due to the split cofferdam elements, the piles are of different lengths, so that the safety against uplift will be nearly the same for both the outer and central cofferdam units during the construction. A total of 46 piles per substructure were built, of which 20 pieces of 19.50 m were installed in the outer cofferdam section and 6 pieces of 20.50 m in the central part. The overall height of the pile cap beam is 4.50 m, from which the underwater concrete varies from 2.50 ÷ 3.50 m depending on the position of the cofferdam element (outer or central).

Fig. 2: Workspace enclosure with steel guard wall

The ascending wall is 8.00 m × 64.90 m at the bottom, and 7.00 m × 63.19 m at the top. The 10.20 m high wall has a inclination of 1:20 on the longer side and ~1:11 at the ends. The nose of the ascending wall was constructed with antifreeze granite stone to prevent damage caused by floating debris and ice drift.

3. PYLON

The pylon is an A-shaped spatial frame structure made of reinforced concrete pylon legs with a box cross-section. The pylon legs are statically clamped structures based on the reinforcement bars extending from the ascending wall. The height from the top of the ascending wall is approximately ~100 m. The overall width of the pylon legs is ~51 m at the starting level.

The construction of the pylon legs was carried out with climbing formwork technology, divided into 29 phases. The height of the building units is 4.07 m in the general section, which is reduced to min. 2.55 m at the anchors of the cables and at the junction corridor.

Fig. 3: Construction of pylon legs with auxiliary structures and cranes

The longitudinal dimension of the pylon bridge decreases from 5.00 m to 3.50 m according to a second degree parabola to a height of 61.18 m from the substructure, then it remains constant. The associated wall thickness is reduced from 1.00 m to 0.50 m. The width of the pylon legs perpendicular to the bridge axis is constant value of 4.00 m, while wall thickness is 0.60-0.50 m. The outer edges of the pylon legs were constructed with a rounding radius of 30 cm.

In the construction state, 40 mm diameter tension rods were installed in the cantilever structure to reduce transverse torques in the outer leg of the pylon legs. The number of tension rods followed the resulting stresses, so they were made with clutch joints at the boundaries of the building units.

In order to reduce the unfavourable impact of the construction state which was worse than the final state, the pylon legs were held together with a steel structure in 2 places besides the tensioning.

In order to make the pylon legs work together, a reinforced concrete beam with a box cross-section was made at a height of ~55 m from the pylon clamping cross-section.

The pylon was built using two cranes. At the northern end of the ascending wall was a 68 m high crane clamped in the substructure without outrigging. On the other side of the substructure, a max. 116 m high crane was built. In order to reduce the stresses in the crane, the crane structure was propped up to the pylon leg at 3 levels.

Fig. 4: Anchoring the tension bar

Fig. 5: Auxiliary structures for pylon construction

The construction of the pylon was statically divided into 7 sections, within which there were several phases. Based on the results of the preliminary calculations, it was essential to develop a construction schedule and a technology that would make sure that temporary effects in the structure would not be less favourable than ultimate stresses. Based on these principles, the first steel support structure was installed at a height of ~33 m from the substructure and a second support structure at a height of ~51.5 m. Not only did the latter auxiliary structure support the pylon legs but it also supported the formwork of the connecting corridor.

By inserting reinforcing brackets the reinforced concrete applied to the pylon leg is supported by a short bracket. The height of the console is 1.35 m, its width varies between 4.00 m ÷ 3.00 m following the width of the pylon. The size of the bracket perpendicular to the bridge axis is 2.25 m below and 1.53 m above because of the skewing of the pylon.

The bearing stools of the bridge structure were constructed with the same design per pylon leg, using 3 different types of bearings. The vertical reaction force between the pylon and the stiffening truss is taken up by a freely movable pot bearing located on the upper horizontal surface of the pylon bracket.

The longitudinal forces from the stiffening truss are absorbed by 2 special hydraulic bearing stools per pylon leg, which are mounted on the vertical end face of the bracket parallel to the bridge axis. The structural design of the bearing stools allows slow movements (e.g. creep, shrinkage, thermal changes), while the effects of braking forces or earthquakes are absorbed by the stool as a rigid support.

To mitigate the impact of earthquakes on the pylon, the hydraulic bearing stools are limited to a force of 2400 kN, so that in the event of an earthquake beyond this value, the bearing stools will release and the stiffening truss will become a longitudinally extending structure suspended only by the stay cables.


Fig. 6: Longitudinal hydraulic bearing stool and a wind shoe

The transverse support of the stiffening truss is provided by a pneumatic wind shoe directly applied to the pylon leg for a span of 1.40 m above the short bracket. By allowing the transverse thermal movement of the stiffening truss, the wind shoes take forces only from the wind load.

The pylon legs and the stay cables are made work together by steel structure anchors installed in the section starting above the reinforced concrete box connecting corridor. The anchoring elements were installed in the intermediate floors of the building units.

Fig. 7: Steel structure being installed in the pylon for anchoring the stay cables

In the pylon legs, a steel staircase was built for traffic on one side and an elevator was provided for access on the other side. The elevator leading up to the level of the connecting corridor is a so-called “funicular” lift due to the leaning of the pylon. Subsequently, a vertical elevator installed in the axis of the bridge can be used for transverse travel in the triangular space with a glass facade formed above the connecting corridor.

Fig. 8: Pylon glass facade and vertical elevator

4. STIFFENING TRUSS

The stiffening truss is made of 51 mounting units with an open cross-section edge beam on both sides. The building units consist of additional individually manufactured load-bearing structural elements: light gage carriageway; box girders; lower cross members; walkway bracket with sidewalk panels and two-sided border brackets.

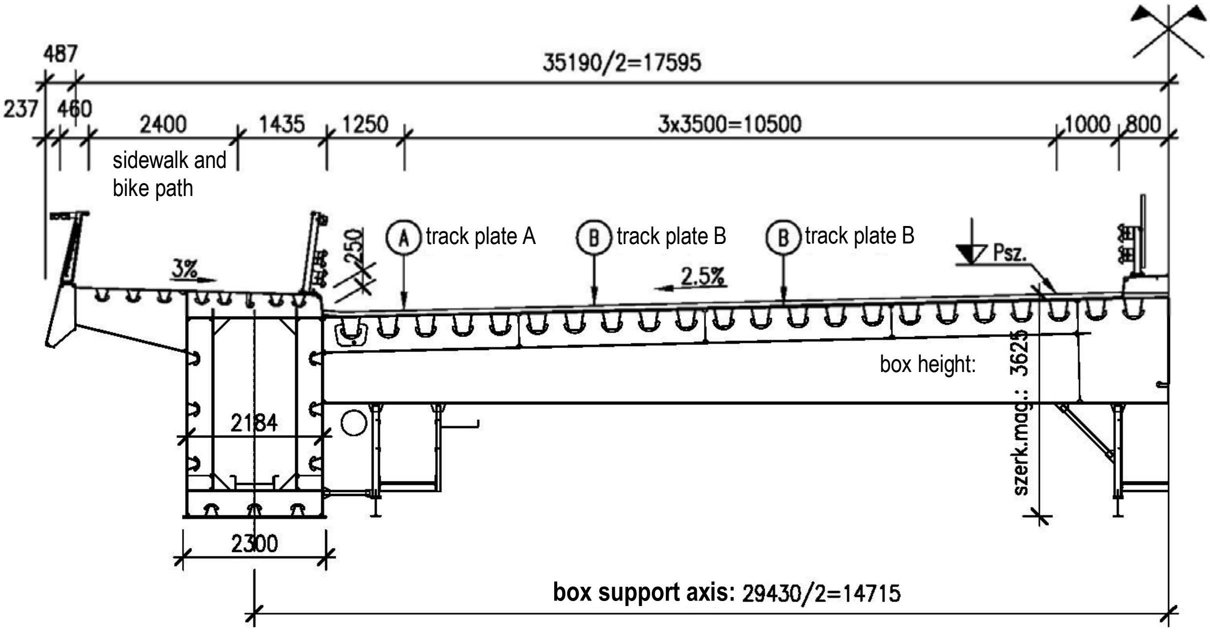

Fig. 9: General cross-section drawing of the stiffening truss

The total width of the steel structure is 36.16 m and the structural height is 3.625 m. The prefabricated individual mounting units were assembled in the pre-assembly plant in Csepel to form full-width mounting units with the lengths of 6, 10 and 12 m. The weight of the units thus assembled ranged from 120 to 160 tons.


Fig. 10: Cross-section of the stiffening truss

The welded orthotropic deck consists of the following 9 manufacturing units in cross section: flat steel track plate, trapezoidal longitudinal rib for longitudinal support, 1 longitudinal support in the bridge axis and cross members every 4 m for transverse support.

The car track slab thickness is 14 mm, and 16 mm at the pylons.

The longitudinal ribs supporting the track slab have a trapezoid cross section and are made 300 mm high and 8 mm thick. Their width reduces from 300 mm to 200 mm. The axis spacing of the ribs is 600 mm, so the distance between the ribs legs is 300 mm.

Also the track plate under the sidewalk has a trapezoidal cross sections but it is supported by a reduced-sized rib and 1 piece of flat steel. Both longitudinal ribs run continuously through the intermediate cross-members.

Fig. 11: Lifting the closing member of the stiffening truss

The section of the cross-members connected to the track plate has a "T" profile and the lower part has an "I" profile. The upper 150×12 mm flange follows the crossing of the track plate, and the lower 400×20 mm flange has a horizontal design, so the 12 mm thick web-plate has a variable height.

The longitudinal ribs are also welded "I" profiles, the upper flange plate is the deck plate, the web-plate has a height of 1398mm and a thickness or 12 or 16 mm, while the lower flange is 300×20 mm that changes to 30 mm at the pylon.

At the edges of the bridge, 14,715 m from the axis of the bridge, box supports are installed, where the lower anchor points of the stay cables are located.

The upper flange of the box support is the sidewalk slab, the bottom is the bottom slab and the sides are made of vertical web-plates. The bottom slab and web-plates are reinforced with trapezoidal longitudinal ribs. The transverse reinforcement also consists of "T" sections. The web-plates have an axis distance of 2184 mm.

The cable connection points are 12.00 m apart in the longitudinal direction. The skew of the cables varies between 26.2° and 65.1° in the longitudinal direction along the bridge axis, and 4.7 ÷ 13.5° in the transverse direction. The active anchoring of the cables is 1.50 m or 2.00 m above the top of the track plate. Thanks to its geometric design, there was enough space inside the box support to tension the cables.

The stiffening truss was mounted with cantilever mounting according to the balance principle. The elements were lifted by the Adam Clark floating crane. In order to reduce the length of the cantilever mounted bracket, an auxiliary yoke support was built in the shore openings, 60 m from the pylon. The closing member was installed on 06/12/2008.

5. STAY CABLES

The stiffening truss is suspended from the pylons by 4x11 cables, totalling 88 cables. The cable lengths range between ~56 ÷ 163 m. The suspending cables were made in parallel from 7-core strands. The material used is of the quality Fp150/1860 specified in design specification e-UT 07.01.14. The cables consist of 31, 37, 55 and 61 strands, depending on the stress. The permissible tension of the cables is 5% according to ÁKMI (2003), considering the reduction in cross-section, the yield strength is 40%.

The cables were stressed below at the anchorage in the box of the stiffening truss. Each strand was stressed using a mono stretching device, with the so-called "isotension" method. Subsequent cable force control is possible in the pylon with an upper (passive) thread-adjustable anchorage, for which a ring press can be used.

The stay cables are equipped with a vibration damper inside the vandalism protection tube at the anchorage at the stiffening truss, at a uniform height of 850mm above the sidewalk level. In line with the expected stresses and vibrations, 3 types of damping equipment are installed. For the shortest cables, IED internal elastomneric dampers, for the two longest cables, IRD internal radial dampers, and for the intermediate 8 cables on both sides, IHD internal hydraulic dampers were installed.

Fig. 12: Types of vibration dampers used

The cables are protected by a double-extruded hard polyethylene casing with a diameter of 160 ÷ 200 mm in the section following the vandalism protection tube. The outer side of the casing is made with double spiral ribs to prevent galloping vibration due to the combined effect of rain and wind.

6. CONSLUSIONS

When designing the structure of the first cable-stayed bridge in Hungary described in the present article, high emphasis was put on the interaction of building technology and building stages. Taking into consideration the steps of construction and their interaction with the structure was inevitable during the design. Continuous consultation and mutual support between the contractor/designer was indispensable for an optimal design both in terms of load capacity and material utilization.

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SMARTPHONE MOTION SENSOR-BASED RIDE QUALITY TEST FOR DETECTING VEHICLE SAFETY AND TRACK MAINTENANCE ISSUES ON TRAMWAYS

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SUMMARY

In this paper, a cost-effective method for monitoring and evaluating the tramway passenger comfort and ride quality is presented using motion sensor data of smartphone fitted to in-service vehicles. Running vehicles experience a broad spectrum of vibrations and oscillations that occur in response to excitation inputs of vehicle-track coupled dynamics. In this work, ride quality is expressed in terms of rail surface defects, track irregularities, and irregular vehicle movements. Data from smartphone built-in sensors such as accelerometer and gyroscope are processed by sensor fusion and are coupled with local and global positioning using GNSS data to identify sections with poor ride quality. Results are promising and demonstrate that poor ride quality can be accurately localized on a tramway network. The proposed method enables infrastructure monitoring done by conventional passenger cars and makes the possibility of comparing the ride quality of traditional and modular designed in-service vehicles.

1. INTRODUCTION

Tramway tracks are guided transportation systems that require several track inspections and examinations to ensure safe operation of the tram fleet. When the vehicle is running on the track, where there are no geometrical deviations from nominal track geometry parameters, the structural elements of the vehicle perform regular motion and follow the alignment of the track. If the track geometry deviates from the ideal, the irregular movement of the vehicle caused by the track defects is accumulated on the previously mentioned regular movement (such as *pitching*, *rolling*, and *yawing*). Vibrations and oscillations of the vehicle caused by track irregularities have an effect on the condition of both the vehicle and the track. In the design process, the values of the track alignment parameters are chosen to ensure a safe riding with at least a minimum comfort level. A good compromise has to be found between train dynamic performance, maintenance of the vehicle and track, and construction costs.

Tramways overcome horizontal curves with much smaller radii than railway vehicles, as well as the passengers in a tramway vehicle are more likely to be standing supported or moving around within the vehicle, therefore the risk of passengers losing their balance and falling is increased (Powell & Palacín, 2015). The standing passengers in tramway transport can have significant lateral force on entering both the small radius curve and the diverging direction of turnouts depending on the vehicle speed and the structural design of the vehicle running gear. However, the limit values of the lateral passenger comfort parameters (*lateral acceleration* and *its rate of change*) are not supported in the present Hungarian regulation by the statistical analysis of the measured kinematic movement characteristics of the new tramway vehicles operated in Budapest.

The attention of this paper is focused on the measurement of tram kinematic movements when entering different track alignment layouts, using smartphone motion sensors fitted to traditional and modular designed in-service vehicles operated in Budapest tram network. To measure the kinematic motion characteristics of these vehicles, we developed an application for android smartphones that is capable of timing synchronized recording of all phone sensor data and GPS location information. During the validation of the recorded data, the physical background of the acceleration, inclination and rotation sensor were verified in the known angular position, rotation and displacement. The calibration tests were returned to the well-known basic physics laws with high accuracy.

After sensor calibration, we determined both the virtual transition length and the representative cross-sections of investigated tramcars in terms of lateral passenger comfort (using the data of yaw-rate gyroscope and lateral acceleration) and then the line tests were carried out only in the relevant vehicle cross-sections.

During the kinematic analysis, the lateral accelerations and the yaw-rate gyroscope data recorded on the car body were investigated. We analyzed the peak values and the main characteristics of the recorded signals. Clearly demonstrated when comparing the different types of trams that modern multi-modular vehicles have grater dynamic impact on the track like the traditional ones in case of unfavourable track geometry.

In the next section, details are given about the limitations of applying standards related to ride comfort evaluation using inertial sensors on tramway operation. *Section 3* introduces the measurement setup adopted for experiments, the newly developed android application “CAFat” for data acquisition and the applied data processing methods. Then, *section 4* is intended to face the line test results and gives the main conclusions on the considered possibilities for the development of the currently used tramway track and vehicle condition monitoring.

2. THE APPLICATION OF STANDARDS REGULATING THE ADMISSIBLE LIMITS OF LATERAL ACCELERATION AND ITS RATE OF CHANGE ON TRAMWAYS

Vibration or oscillation caused discomfort influenced by several factors including local constraints, vehicle condition, and track quality, therefore there are no universally applicable standards to quantify the ride comfort (Munawir et al., 2017).

In the track alignment standards, the transition and circular curve parameters at certain design speed are chosen according to the limit value of non-compensated lateral acceleration and its rate of change (h-vector). For the standing passengers in the tramway vehicle it is needed to provide the best travel comfort, so keep these ride comfort parameters at the minimum. If the value of the h-vector is very strict, large transient curve lengths should be planned, which may not always fit on the urban area due to the constraints and the limited space available. In this case, designers either decide to reduce the design speed (which is not recommended) or request an exemption from the authority. Although the Hungarian Regulations like OKVPSZ (KPM, 1983) or BKV “Yellow Book” (BKV, 2000) contain limits for kinematic motion characteristics, they do not specify their exact measurement conditions or the data processing methods.

Among EN standards it is important to mention the EN 12299:2009, which clarified the measurement process, place and method of evaluation as well. The standard also specifies the method for determining the representative vehicle cross-section and the height position of the measurement (in the case of a seated or standing passenger). The measurement conditions described in the above standard are only applicable to conventional railway vehicles. Due to the different characteristics of the conventional and light rail vehicles, the regulations in the standard can be used only as a guideline on tramways.

3. APPLIED MEASUREMENT SYSTEM USING MOBILE PHONE SENSORS

3.1. "CAFat" Data acquisition app

Today's smartphones have several sensors whose data can be used to infer the vehicle kinematics characteristics. We developed an application called "CAFat" running on the Android operating system, whose output data can be quickly processed using any office program.

The program has a "Start" button that starts the measurement, which changes to "Stop" after measurement begins. It also shows the time elapsed since the beginning of the measurement and the accuracy of GPS location data.

The recorded data is written in a "CSV" file. The first and second row of the measurement file shows the *beginning* and *end date of the measurement*, as well as the *sampling frequency*. The software can access the sensor data in the phone read out at the maximum sampling rate. Depending on your phone model, you may vary from 100 Hz (for mid-range phones) to 500 Hz (for premium-category phones). To determine the irregular vehicle movements, the 100 Hz sampling is sufficient, because the car body oscillations between 0 and 20 Hz affect only the non-compensated lateral acceleration. Higher frequency components cause only vibration. The measured parameters are listed below:

- The first parameter is the elapsed *Time* t [s]: since more than 100 samples are taken in every second, the time is expressed in millisecond accuracy;
- *Vehicle Speed* v [m/s]: displays the current speed from the GPS data at every second;
- *Latitude and Longitude coordinates*: the exact location of the measurement in the geographic coordinate system;
- *Accuracy of GPS location data*: this parameter shows the accuracy of the positioning in meter unit;
- *Three-axis accelerometer sensor data*: the accelerometer is capable to detect the sudden impact acting on the 3-axis and the 3D orientation in a motionless position. In this research, it helps to detect lateral acceleration, that is, the main influencing factor of transverse comfort.
- *Three-axis Inclinometer data*: "*pitch*", "*roll*" and "*azimuth*" can be divided into the following irregular vehicle movements: rolling, yawing and pitching, providing the angle position of the three measuring axes in degrees. The interpretation of the axes is illustrated in Fig. 1.
- *Three-axis gyroscope data*: the recorded parameters "*gyro_x*", "*gyro_y*" and "*gyro_z*" return the angular velocity for all three axes. The smartphone uses a gyroscope for

horizontal or vertical adjustment of the screen, or in the 3D motion-controlled games to improve the gaming experience.

- *Magnetometer*: The parameter "*deg*" shows the deviation from the magnetic north in degrees, allowing the relative positioning of the device without location data. This allows you to get information about movement even if the GPS signal is lost or if the accuracy of the positioning is incorrect.

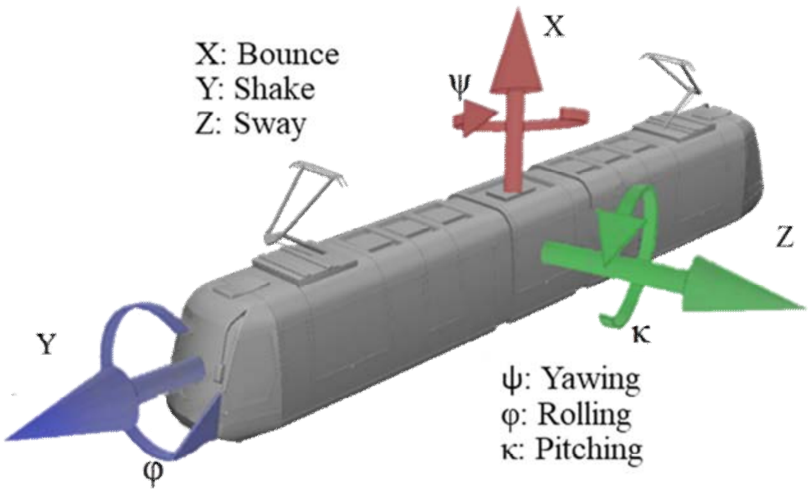

Fig. 1: Interpretation of measurement axes and the irregular vehicle movements (pitching, rolling, and yawing)

Fig. 1 shows the interpretation of the measurement axes, which enables to identify the spatial irregular vehicle movements using the measurement setup introduced in the next section.

3.2. Measurement setup

The axis arrangement of sensors in smartphones is standardized, independent of manufacturer. Accelerometer, inclinometer and gyroscope are suitable for describing spatial motion (3 measuring axes) and the measuring axes follow a right-handed coordinate system. If the phone is positioned horizontally with a display facing upwards so that the charger connector is closer to you, the '*x*' axis is to the right, the '*y*' axis along the long side of the phone towards the camera, and '*z*' the positive direction of the axis points vertically upwards. The positive direction of rotations around these axes is also determined by the right-hand rule. During the measurements, the phone is placed on its long side fixing to the wall of the car body so that the positive direction of the '*y*' axis is the same as the travel direction. In the measuring arrangement used, the axis '*y*' records the longitudinal acceleration, the axis '*x*' is the vertical acceleration, while the axis '*z*' records the lateral acceleration of the car body. The pivoting around the '*x*' axis is the yaw movement of the vehicle, the movement around '*y*' axis is rolling, and the pitching movement is around the axis '*z*' (Fig. 1). For most of the measurements, the device was attached to the vehicle window using a silicone pad, thus preventing the motion of a human error to produce poor results.

3.3. Data processing

The most important data in the evaluation is the yaw-rate angular velocity, which means a rotation about the vertical axis. This data accurately describes the steering mechanism of the vehicle on curved track sections and not sensitive to the rolling of the vehicle. For each evaluation, a 2 Hz low-pass filter was used uniformly on the raw gyroscope data that complies with the relevant requirements of the MSZ EN 12299 standard.

It is important to emphasize that by eliminating the effect of the additional load resulting from the car body rolling; it is reduced to a lower value compared to the real values. Quasi-static values are obtained. However, both domestic and foreign regulations only set limit values for quasi-static values, so the quasi-static lateral acceleration produced from the gyroscope data was used as the basis for the comparison with the theoretical values.

A.) Processing and interpreting acceleration data

The device vibration, sudden impact shocks and the orientation in a motionless position can be sensed with accelerometers. The acceleration data measured on the car body is extremely noisy and can not be used directly, prior processing is required. In order to remove vibrations of higher frequency components that are irrelevant for the kinematic test, a two-way moving average method is applied using the equation (1):

$$\widehat{a}_t = \frac{1}{2k+1}\sum_{j=-k}^{k} a_{t+j} \qquad t = k+1, k+2, ..., n-k \tag{1}$$

where k: the number of members used to calculate the moving average,
$2k+1$: Number of moving averaged sections,
$\hat{a}_t$ [m/s^2]: The calculated moving average,
a_t [m/s^2]: Vibration acceleration on the car body. $a_t = a_1, a_2, a_3, ..., a_n$, where n is the total number of elements in the recorded signal.

B.) Processing and interpreting gyroscope data

The gyroscope is suitable for describing irregular and oscillating movements of the car body. The track alignment and the car body tilt of the train can be derived from the data of yaw-rate and roll-rate gyroscope data respectively.

During measurement, the yaw-rate gyroscope data highly influenced by the design of the vehicle running gear design. The steering of bogie vehicles into the curve takes place during the virtual transition length. Fig. 2 illustrates the curve sensing of a traditional two bogie vehicle with pivot distance (d = 6 m) travelling at a constant speed, which clearly shows that the vehicle starts to detect the curve 6 m before the beginning of the curve. The virtual transition length of vehicles operated in Budapest is significantly different (see section 4.1).

C.) Relationship between lateral acceleration and yaw-rate gyroscope data

Fig. 3 compares lateral acceleration and the yaw-rate gyroscope data recorded by Xiaomi Pocophone F1 on tram line 49 between Szent Gellért tér and Fővám tér. The top chart shows the velocity recorded by GPS, the second one introduces the raw (grey-coloured graph) and the filtered gyroscope data (black-coloured graph), while the third one shows the non-filtered (grey) and filtered (black) lateral accelerations. During the measurement the Ganz type, articulated tram was investigated, which has poor vibration properties.

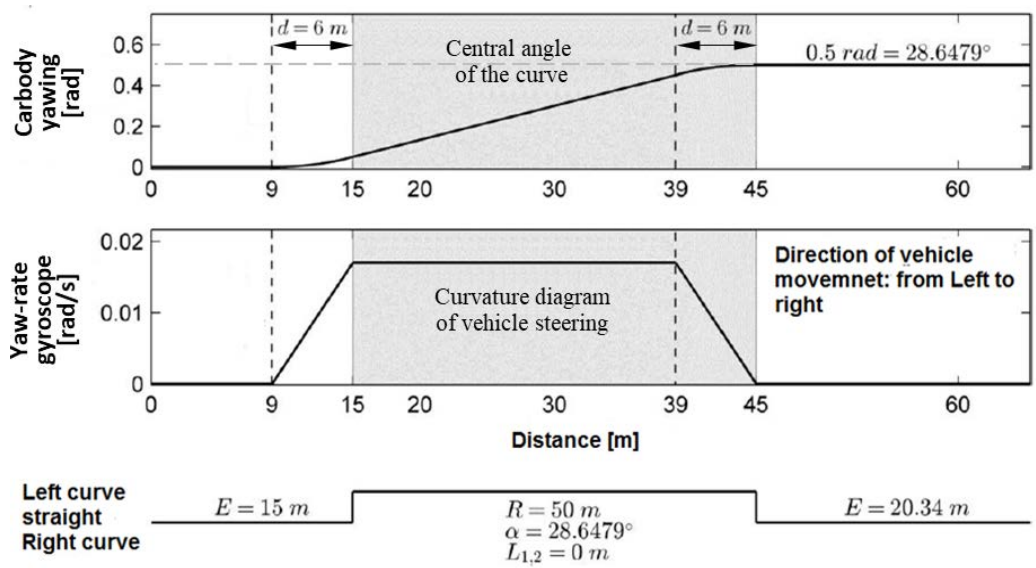

Fig. 2: The interpretation of yaw-rate gyroscope data and virtual transition length (d = 6 m) of a traditional two bogie vehicle steering into a circular curve without transition curve

The track curvature can be obtained from the filtered raw acceleration data (using a 0.5 Hz low-pass filter or a moving average using 1.00 s windows width, see the third graph in Fig. 3), while in the case of the gyroscope, the raw data clearly contain this information. It is important to mention that when examining vehicles with good running properties, the raw acceleration data also clearly shows the horizontal curves.

Filtered lateral acceleration and gyroscope data are highly similar (see graphs 2 and 3 in Fig. 3), which is due to the fact that the data provided by the gyroscope can be used to calculate the quasi-static lateral acceleration using the equation (2):

$$a_0 = v \cdot \omega = v \cdot \frac{v}{R} = \frac{v^2}{R} \tag{2}$$

where a_0 [m/s^2]: quasi-static lateral acceleration,
v [m/s]: velocity,
ω [rad/s]: angular velocity,
R [m]: radius of the curve.

Furthermore, the car body tilting acceleration can be computed by the difference between the filtered lateral acceleration and the calculated quasi-static lateral acceleration. On the section of "Szabadság híd" (between the position of 3300 and 3600 m) the track is built with 22 mm superelevation. Fig. 3 clearly shows that the yaw-rate gyro is not sensitive to car body roll, while the recorded lateral acceleration contains both the tilting-, and the centrifugal acceleration. The fourth graph on Fig. 3 shows the calculated tilting acceleration compared to the nominal value of track cant measured by TrackScan track geometry measuring trolley (red-coloured graph). The reference tilting acceleration is calculated from the measured track cant value using the equation (3):

$$a_{tilt} = g \cdot \frac{m}{t} \tag{3}$$

where m [m/s^2]: track cant,
g [m/s]: gravitational acceleration,
t [mm]: track width (1500 mm),

D.) Sensor fusion between magnetometer, accelerometer and the gyroscope data

To calculate device absolute orientation sensor fusion is applied. Generally, the accelerometer and magnetometer outputs include a lot of noise. The gyroscope in the device is more accurate and has a very short response time. Its downside is the dreaded gyro drift, which is accumulated when making the sum of angular velocity to get actual orientation.

To avoid both, gyro drift and noisy orientation, the gyroscope output is applied only for orientation changes in short time intervals, while the accelerometer data is used as support information over long periods of time. This is equivalent to low-pass filtering of the accelerometer and magnetic field sensor signals and high-pass filtering of the gyroscope signals.

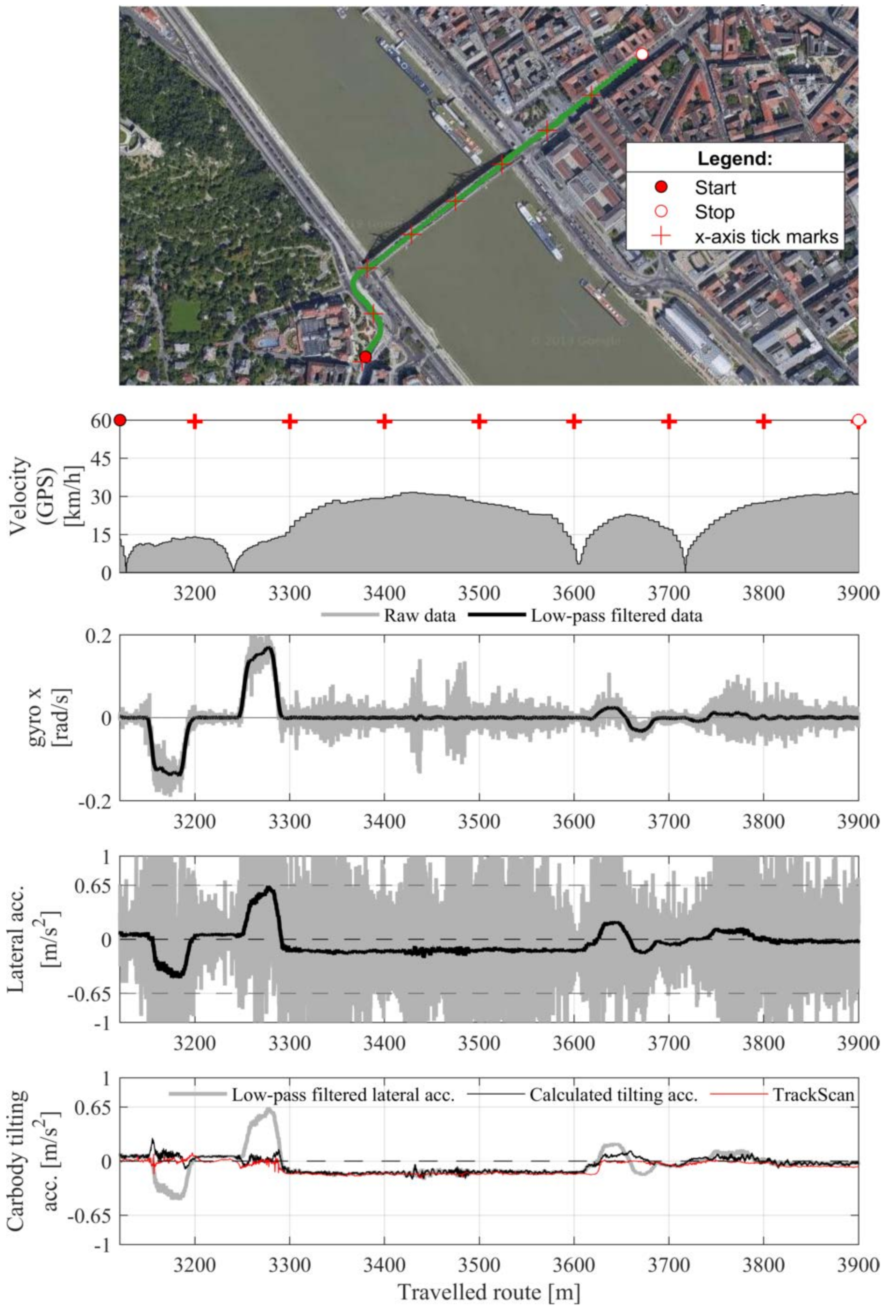

Fig. 3: Determining the tilt of the car body using roll-rate gyroscope and lateral acceleration data of Xiaomi Pocophone F1 fixed to the rear part of the Ganz ICS tram (Tram line 49, Szent Gellért tér – Fővám tér)

4. MEASUREMENT RESULTS

Line test measurements were performed under real traffic conditions with passengers on different classes of tramway vehicles. The current fleet of vehicles operated in Budapest's tram network is not considered homogeneous, apart from standard vehicle configuration, i.e., a car body on two bogies, in modern tram designs various arrangements are applied. The running gear of the modular designed low-floor trams are based on a highly sophisticated axlebridge component and integrated completely into the car body. This structural design produces significant additional stress for the track compared to the traditional bogie vehicles.

The primary purpose of the tests was to determine the ride comfort, therefore the representative position of the passengers was decisive when selecting the measurement location within the vehicles. In determining the critical cross-section, several cross-sections were simultaneously measured on a vehicle and then the line tests were performed only in the determined relevant cross-section.

During the selection of measurement places for line test, the primary consideration was to quantify the extent of irregular vehicle movement (lateral sway and oscillation) and to search for a relationship with the track alignment parameters.

4.1. Curving behaviour of investigated tramcars

The virtual transition length of the vehicles was investigated on entering circular curves or on a reverse curve with intermediate straight section. In the case of abrupt change in curvature, the approximate formulas for determining the lateral acceleration do not take into account the virtual transition length of the vehicle, so curvature function is defined containing both the nominal track alignment parameters and curving behaviour of the investigated vehicles similarly as shown in the Fig. 2. This curvature function must be matching line to the filtered yaw-rate gyroscope data. It is important to note that the value of the virtual transition length determined during the measurements depends greatly on the accuracy of the velocity data, from which the travelled distance was calculated. Nevertheless, the curving behaviour of the full vehicle and its parts or modules could be properly separated. The virtual transition length of vehicles validated by measurements is summarized in Tab. 1.

Tab. 1: Virtual transition length (d) of tram fleet operated in Budapest

CAF Urbos3 (5 module)		CAF Urbos3 (9 module)		Siemens Combino		TATRA T5C5	TW6000	Ganz ICS
Module	d [m]	Module	d [m]	Modul	d [m]	d [m]	d [m]	d [m]
C1, C2	1,800	C1, C2	1,800	1-6	1,800	6,700	6,400	6,000
S1, S2	6,745	S1-S4	6,745					
R2	1,850	R1, R2	1,850					
		M	1,800					

Wheelbase; bogie pivot distance; module length

The virtual transition length of traditional bogie vehicles and articulated vehicles equal their pivot distance. However, the curving behaviour of modular designed low-floor trams can vary per modules according to the design of their running gear. In the case of Combino tram, each module is supported by axlebridge and its virtual transition length equal to the wheelbase (1,80 m). The CAF vehicle consists of suspended (non-folded) and driven car body parts. The length of the virtual transition that significantly affects the curving behaviour varies by vehicle modules: wheelbase distance for driven modules, and module length for suspended ones. Due to the different virtual transition length, the most sensitive parts of the vehicle are the front and rear modules.

Fig. 5 compares the curving behaviour of a conventional bogie and modern low-floor vehicle based on the angular rotation about the vertical axis measured on the car body. The blue-colored diagrams show the low-pass filtered yaw-rate gyroscope data ($gyro_x$) of CAF and TATRA trams respectively, while the black graphs show the defined curvature function, which contains the identified virtual transition lengths of the vehicle or investigated module. In both cases, the measurements is performed at the rear of the trams using Samsung Galaxy S8 mobile phone. The irregular vehicle movement of modular low-floor CAF trams in low-radius curves without transition is clearly evident.

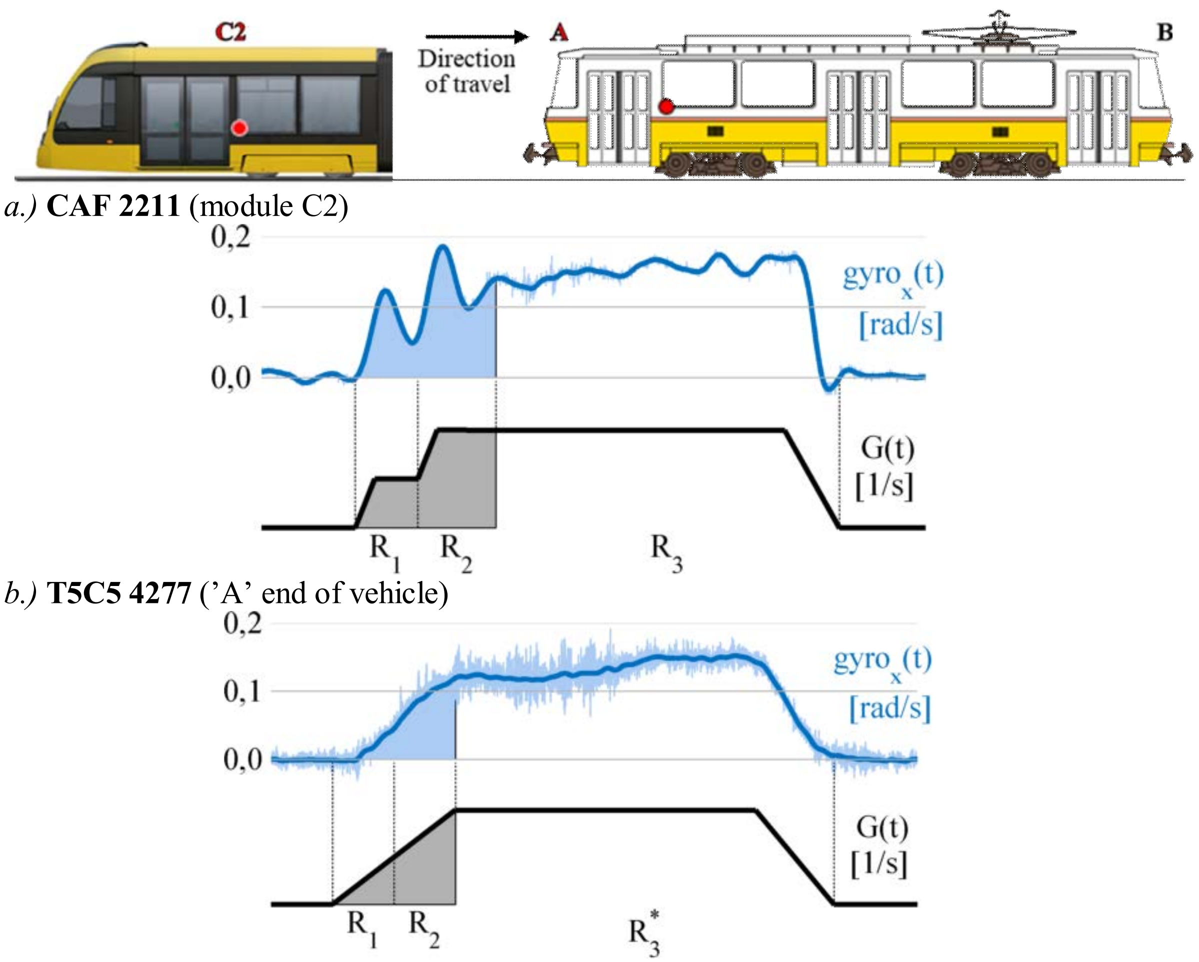

Fig. 4: Comparative analysis of a.) CAF and b.) TATRA T5C5 vehicles curving behaviour at same velocity on VPh50/25 single track crossing turnout and connecting curved track using yaw-rate gyroscope data of Samsung Galaxy S8 fixed to the rear part of the trams (Tram line 17, Margit híd, budai hídfő)

4.2. Quantify dynamic vehicle response on track irregularities

The design of vehicle running gear has an important influence on the vehicle response when it is entering track irregularities. The dynamic vehicle response of traditional and low-floor trams are tested on lateral track irregularities, which is illustrated in Fig. 5.

Fig. 5: Investigated lateral track irregularities (on tram line 4 and 6, at Mechwart liget)

The travelled rout is calculated from the GPS velocity data and compared to the nominal track geometry parameters. The differences were less than 1 m in every tested section. The 'tolerance to track defects' of the vehicles operated on this curved track section were compared with each other. Fig. 6 shows the following diagrams: vehicle velocity, yaw-rate gyroscope data of CAF, TATRA, COMBINO trams describing their curving behaviour, and curvature function of the track. The diagrams also show the schematic representation of the vehicles and the place of the measurement. All measurements were performed twice. The track defect disrupted the running of every vehicle tested, but the extent and nature of irregular vehicle movement varied across vehicle types. In the case of CAF tram, the largest lateral shake was measured, its period and wavelength being nearly the same as the wheelbase of the tested vehicle module. In the case of the Combino, we detected irregular vehicle movements influenced by the wheelbase distance, but this vehicle showed double "waves". This phenomenon is explained by the hydraulic control unit integrated into the Combino modules, which allows the bogies of the connected modules to turn only at the same angle. In the case of the TATRA tram, the sway caused by the track defect is significantly damped, the wavelength and the period of movement are approximately equal to the pivot distance of the vehicle. Overall, the CAF and the Combino trams less tolerant to track defects compared to conventional bogie vehicles. The irregular vehicle movement due to repeated passage on track defects can cause early deterioration of the track.

4.3. Estimating the effectiveness of rail maintenance work

The rail corrugation caused vibration can be sensed on the car body and if the rail is maintained, this vibration will be significantly reduced. This changes can be accurately quantified by using inertial sensors of car body mounted smartphone if the instrumented vehicle condition and the measurement speed are the same before and after intervention work on the investigated track section. The efficiency of rail maintenance interventions is illustrated in Fig. 7, which shows the following diagrams: vehicle velocity, yaw-rate, and roll-rate gyroscope data and the alignment parameters of the track. Measurements were made before and after rail grinding on in-service CAF tram's C2 module. The velocity profile was the same during the measurements, which enables to compare correctly the measurement results. It is clear that after grinding both yaw-, and roll-rate gyro (red-coloured graph) data significantly reduced and the remained faulted rail welds can be easily periodically identified.

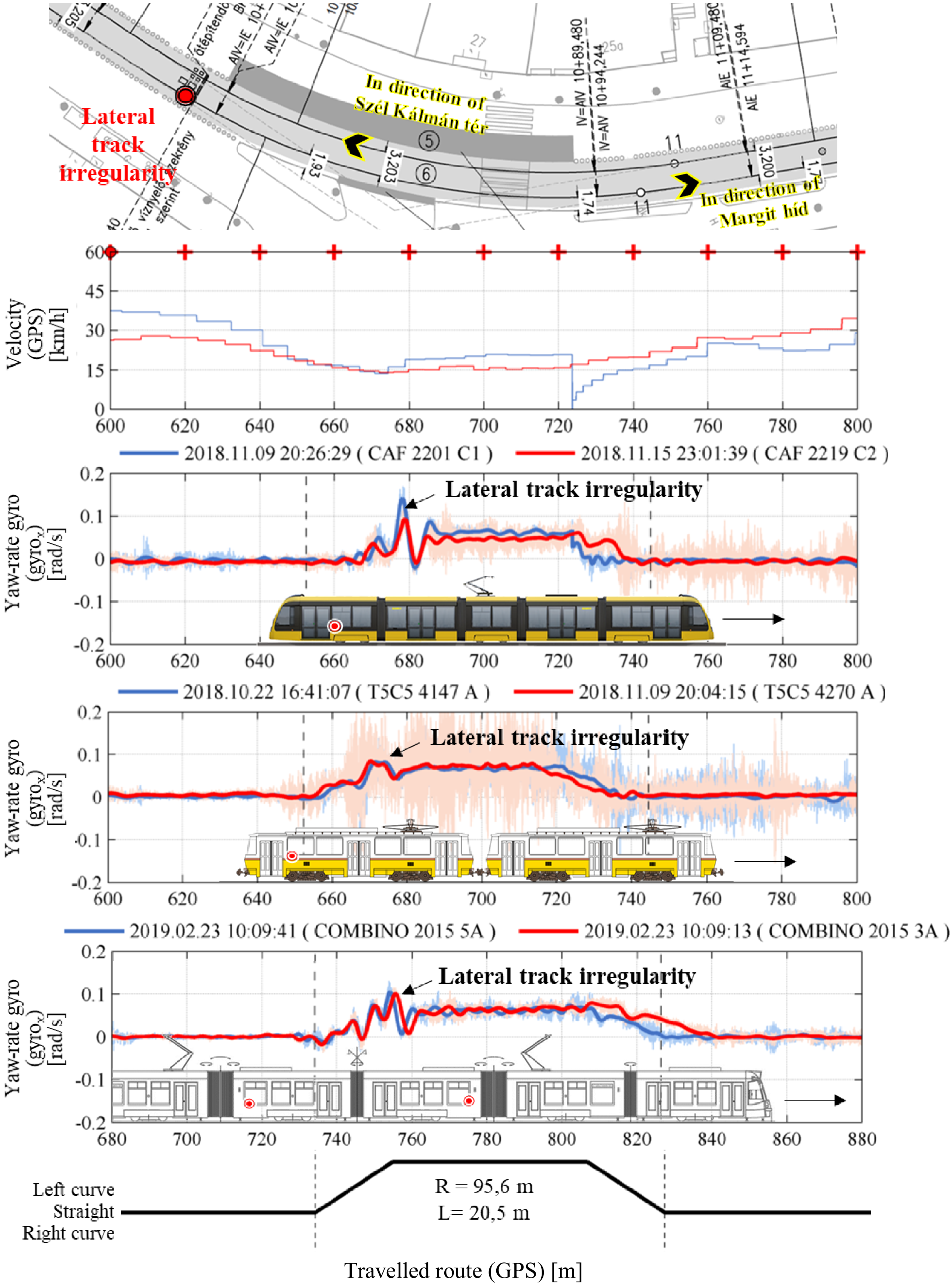

Fig. 6: Dynamic vehicle response of CAF, TATRA és Siemens Combino Supra on lateral track irregularities in curved track with transition curve (Samsung Galaxy S8)

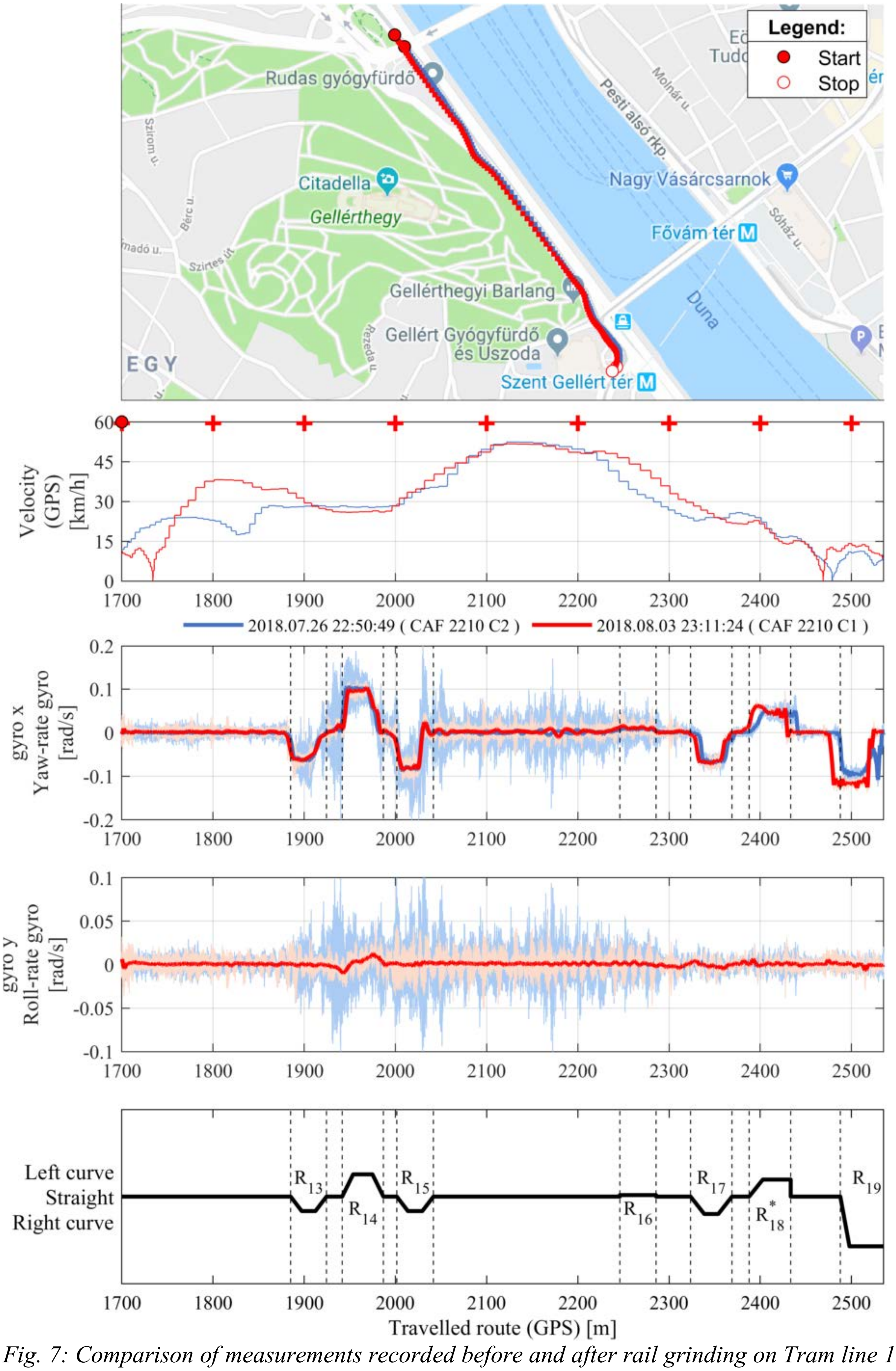

Fig. 7: Comparison of measurements recorded before and after rail grinding on Tram line 19 between Döbrentei tér and Szent Gellért tér (Samsung Galaxy S8)

5. CONCLUSIONS

This paper presents measurements of tram kinematic movements and vibration using Smartphone Motion Sensors, as well as qualifies the curving behaviour of different classes of tramway vehicles and estimates the vehicle dynamic response on track/rail irregularity. The performance and sensitivity of sensors in high-end smartphones have significantly improved over the past few years, enabling them to provide useful information to track management due to their reliability and lower cost compared to industrial solutions. The accuracy of the high-end smartphones built-in sensors influenced by many factors (running gear design, vehicle condition, velocity) and only partially ensure the "mm" precision required for the railways industry, but the real-time measurement of vehicle vibrations during commercial operation allows for rapid response, such as emergency track inspection and maintenance, in situations when vehicle vibration observations detect irregularly large deviations from standard control values (see Fig. 7). Thus, the use of this monitoring system to perform continuous monitoring of vehicle vibrations allows early detection of deterioration or other track irregularities, thus enabling railway operators to conduct effective maintenance work.

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THE CONCEPT OF PERMANENT NOISE MONITORING STATIONS ALONG HIGHWAYS AND RAILWAYLINES

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SUMMARY

Decibels and noise maps, data and colourful posters are untapped assets – they don't reach their final destination unless proper communication. This paper presents a unique concept a combined technical and communication approach. The case-study we examine is the practical application of long-term monitoring system to support infrastructure operations. On the one hand we show the benefits of noise mapping and a simplified noise indicator for communication. On the other hand, this paper presents a unique approach, the practical application of long-term monitoring system to support infrastructure operations and the well being of residents. If not used properly, the output of a long-term noise monitoring system is limited to numeric values without leading to any conclusions. To avoid this common failure, the authors established the future goals of the system long before installing it. The main goal providing information on the noise situation for a) stakeholders of the govt. b) infrastructure operators c) residents. Every group needs different ways of communication to understand and make use of the results. The presented concept delivers solution to address all three groups. Tools like simplified NPI or the developed dynamic noise map all automatically generated by the system serve awareness raising purposes.

1. INTRODUCTION

Without clear explanation, lot of expert work done is useless. Decibels and noise maps, data and colourful posters are untapped assets – they don't reach their final destination: the residents, the stake-holders, the infrastructure designers and operators. The key solution is communication: direct communication or indirect. Personal presence and addressing through the tools of modern technology. To be effective, both solutions must be used in a process.

This paper presents a unique concept a combined technical and communication approach. The main goal providing information on the noise situation. For that purpose, we need data, noise indicators, and a representative demonstration. A noise map clearly shows the different levels of noise pollution. Difference noise maps show the extent of change. A good noise map is illustrative, it makes the noise propagation understandable as well as the noise reduction. However, noise maps are based on historical data. In this paper we present the correct use of real time monitoring data to support general understanding.

2. LONG-TERM NOISE MONITORING SYSTEM AND COMMUNICATING

2.1. Preparations before the installation of the system

2.1.1. Long-term goals

If not used properly, the output of a long-term noise monitoring system is limited to numeric values without leading to any conclusions. To avoid this common failure, it is recommended to establish the future goals of the system long before installing it. The main goal providing information on the noise situation for a) stakeholders of the govt. b) infrastructure designers or operators c) residents. Every group needs different ways of communication to understand and make use of the results. The system must be designed to address those individual needs.

2.1.2. Contribution to (infrastructure planning)

Results of noise monitoring is a good tool for noise reduction if they are part of long-term noise control campaign, where the outcome is linked to infrastructure design. The monitoring results provide baseline for future developments, e.g. noise insulation or traffic on the road network should be designed to protect the well-beings of residents.
Locations of new sensitive developments should be selected based on the results of the noise monitoring network. Areas having excessive noise levels should not be selected for residential developments, only for industrial or commercial purposes.

2.2. Assessment of the monitoring results

2.2.1. The developed web-based application

Presentation of the results is a key factor in the success of the presentation of the results. A two-fold system was designed and implemented for this purpose. Detailed and raw data were made available only for authorized personnel whereas general results are shared with the public.

Besides the common terminology employing LAeq, Lmax, Lmin, etc. a Noise Pollution Index was developed and calculated using a web based application. The Noise Pollution Index (referred as NPI) is based on the well-known Harmonica Index developed during the HARMONICA project in the EU(1). The index is tailored to suit local conditions. The application is developed to display the NPI and to provide better understanding of the noise situation to the public. The index, based on the Harmonica Index, consists of two components a component related to background noise (marked with BGN) and a component related to events, representing the noise peaks (marked with EVT). Adding these components together leads to the NPI which has a scale between 0 and 10.

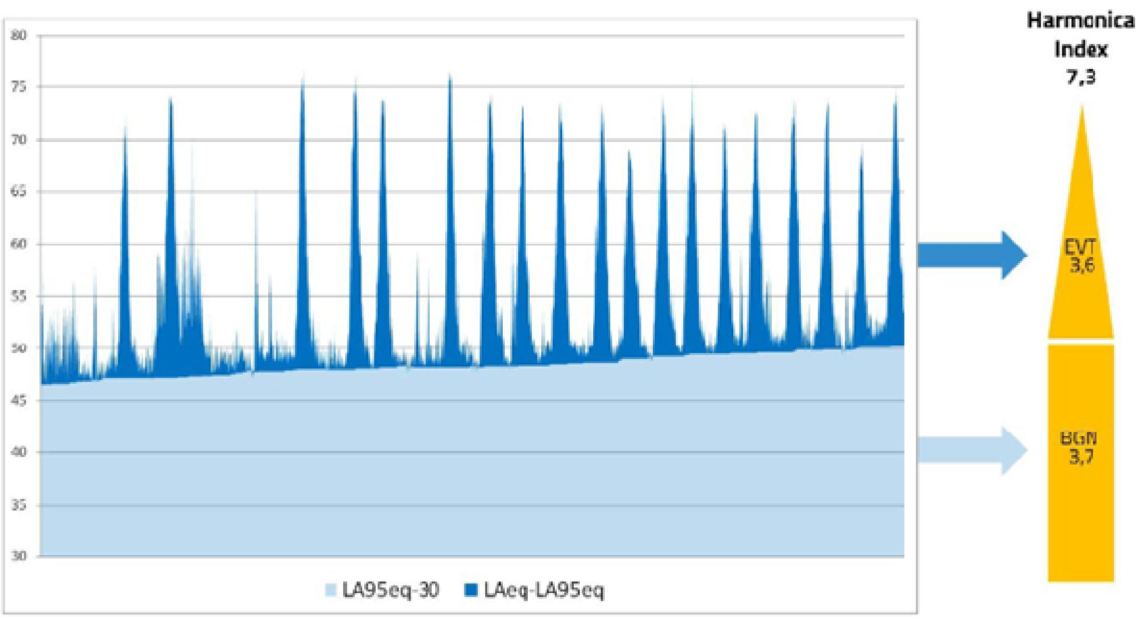

Fig. 1: The Noise Pollution Index

2.2.2. Key performance indicators

Besides the parameters and dynamic noise map key performance indicators (referred as KPIs) have been assessed to provide more understandable values, however they have to be suited to be representative for local conditions.

LKZ (LärmKennZiffer)
LärmKennZiffer(2) (LKZ) describes the effects created by noise exposure in a road. It was utilized in many action plans in Germany and throughout the EU. It combines the noise exposures in the road with the number of people affected. The assessed noise indicator is a product of exceeding a limit value of noise disturbances and the people affected, it is high in places where high residential density and high noise level come together.

Highly annoyed (%HA)
Studying this indicator(3) synthesis curves were prepared for noise annoyance from aircraft, road traffic and railway noise, with 95% confidence intervals taking into account the variation between individuals and studies. These curves were based on studies examined for which Lden (and Ldn), and the percentage of "highly annoyed" persons (%HA) meeting certain minimal requirements could be derived, augmented by a number of additional studies. The percentage of "highly annoyed" persons (%HA) was defined as a function of noise exposure indicated by Lden.

2.2.3. Use of the results for infrastructure design

Proper infrastructural and architectural design can help to reduce the effects of noise. The noise levels inside buildings can be reduced with different passive methods such as noise abatement windows, building walls with high insertion loss. Another way to reduce noise conflicts is to have proper traffic planning. However, those tools can only be applied cost-effectively if valid and reliable input data is available.

Each of the stations' results is specific for an evaluated area. Dynamic noise maps that synchronize with noise monitoring and traffic data illustrate the noise load change, its daily flow, and the effects of altered traffic or extraordinary events. These results can be made available for infrastructure designers and operators to achieve the needed noise comfort in their developments.

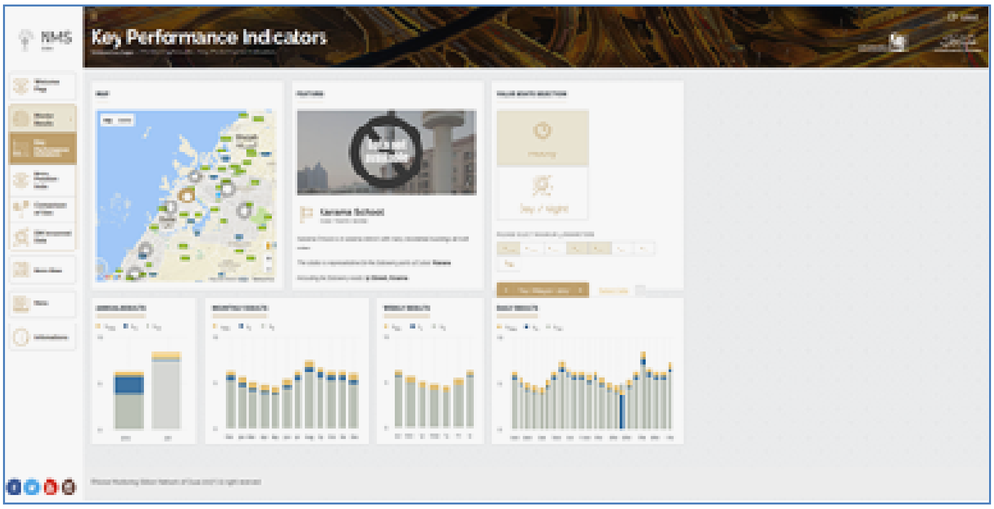

Fig. 2

2.3. Communication concept

Working with an asset like simplified NPI or the developed dynamic noise map requires lot of communication. At design preparatory phase before decision and after, during design phase with public involvement. Communication is inevitable and it is a priority before and during official proceedings. The ways of providing information are different depending on the target group. Decision makers - stakeholders of the govt. can be caught to versatile information. What they need to see in order to effectively involve them in the process: opening up economic benefits, political communication channels to people, giving a vision of a mission. They do not have to understand the professional context.

Decision makers need a set of data, e.g. long-term noise monitoring system, quantifiable goals that can be realized. At the same time, the visual experience like the dynamap makes the point convincing. To do this, the communication path presentations, reports - data warehouse and advantage-benefit recommendations.

Infrastructure designers and operators need a direct indication of how much noise is caused and what can be done to cost effectively mitigate. Data that influences cost planning, spatiality that makes the dimensions of noise acceptable.

For this, the communication path, besides data and reports, is the effectiveness of personal involvement. We speak one language, rowing in the same ship.

3. CONCLUSIONS

Noise is becoming a more and more severe issue globally. The authors of this paper are pioneers to handle environmental noise on a strategic level. Conducting awareness raising campaigns and designing strategies, action plans.

Solutions such as noise barriers are yet to be accepted by the public and decision makers, but by utilizing road design and operation as a major tool for noise mitigation, measures can be taken to significantly improve the noise situation and quality of living. The noise monitoring system, the evaluation and the presentation of the results as described in this paper represent a milestone in delivering unique noise data to support road designers and operators to achieve the needed acoustic comfort.

GYSEV 8 RAILWAY LINE, COMPLEX DEVELOPMENT PLAN OF CSORNA'S RAILWAY STATION

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SUMMARY

Despite Csorna is not an administratively important settlement, the town's railway station is one of the most important centre of the passenger and freight railway transportation in north-western Hungary.

The reason for this: there are two major railway lines and one with less traffic with intersect only here. In recent decades the station could not keep up with the evolving needs because of the increased traffic and the regulations.

During the preparation of my Master's thesis, I prepared a development study plan in the light of the long-term goals and capacity increase which help the significant railway traffic with it's easily passage. The intervention meets not only the expected needs locally, but also in harmony with the neighbouring stations, taking into account the whole railway line. In addition the planned station complies with applicable regulations.

1. PRESENTATION DESCRIPTION OF CSORNA'S RAILWAY STATION

The subject of my thesis was the development of railway station in the town of Csorna, which is located in the central part of Győr-Moson-Sopron county in the micro-region of Csorna. The two main routes in the area are: main road 85 and 86. Most of the area belongs to the Fertő-Hanság National Park, and the landscapes here is very diverse. The whole area characterized by small villages, with traditional village settlement structure. Csorna is the only one settlement with city rank, so this is the centre of the micro-region. This is where the region's 30 % of population lives: 10,400 people. The settlement centre offers unique value to both environmental features, also architectures and traditions.

The subject as before of my work is the train station, which is 1.1 km from the town centre and operated by the Raaberbahn AG. (GySEV). The joint use of the station with MÁV (Hungarian Railways) is regulated by a framework contract.

1.1. Station intersections

In Fig. 1 can be seen the railway connections and the location of Csorna's railway station. The station is located next to the following railway lines:

- Győr – Sopron railway line (GySEV 8) between Kóny and Rábatamási stations,
- Hegyeshalom – Porpác – Szombathely railway line (GySEV 16) between Szil-Sopronnémeti and Bősárkány stations,

- Csorna – Pápa railway line (MÁV 14) where Csorna is a final destination and the neighbour station is Egyed-Rábacsanak.

Fig. 1: Csorna's railway connections

1.2. Characteristic of the railway station

The station characteristic is absolutely complex and has many aspects. Currently it operates as a district control centre.

Csorna's railway station is a(n):

- junction station for line 16 and 14;
- interchange station for line 8;
- departure station for line 8 (InterCity trains - IC) and line 14 and 16 (stopping trains and express trains);
- departure station for freight trains departing from here;
- opened station for unlimited passenger and freight transport services;
- flat marshalling and interlocked location.

The district control office is located in the KÖFI (Central Traffic Management Office) where not only Csorna, but also more other neighbour stations are controlled (can be seen in Fig. 2).

1.3. Neighbouring stations

The neighbouring stations of the railway station are as follows:

- Kóny /No. 8 railway line to Győr/;
- Farád /No. 8 railway line to Sopron/;
- Rábapordány /No. 14 railway line to Pápa/;
- Szil-Sopronnémeti /No. 16 railway line to Szombathely/;
- Bősárkány /No. 16 railway line to Hegyeshalom/.

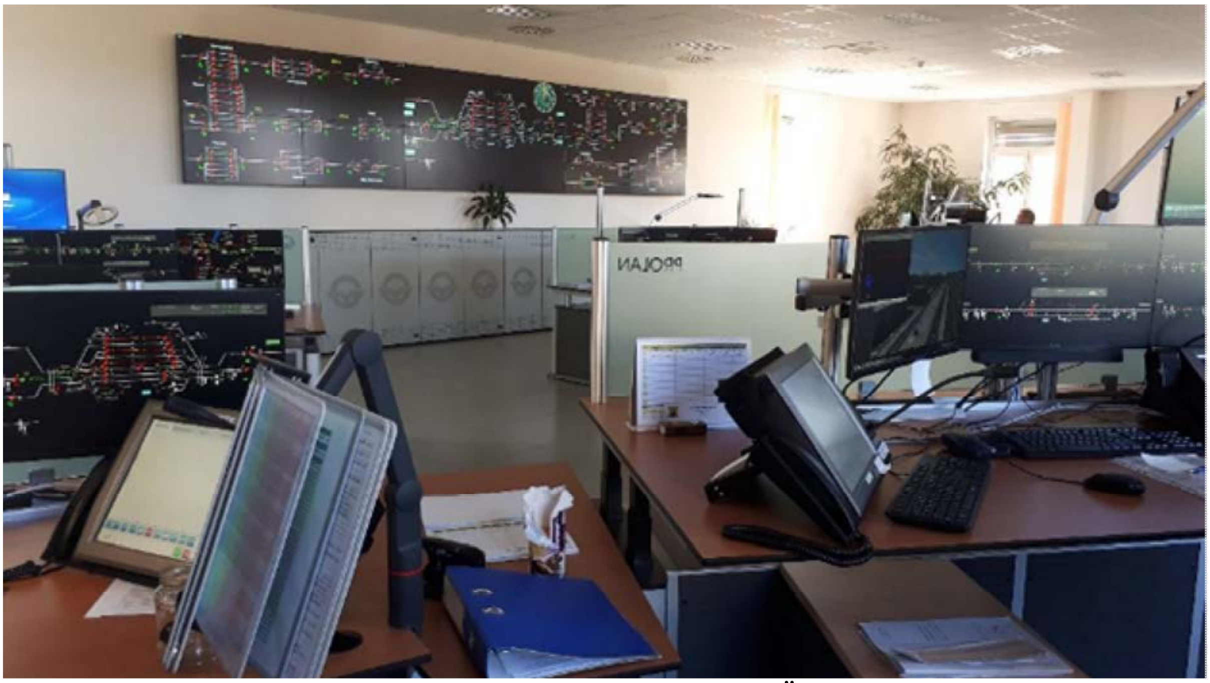

Fig. 2 :Huge displays in the KÖFI

1.4. Description of tracks and trains passing by

The station has eight railway tracks for arriving trains (2, 3, 4a, 5, 7, 8 and 9). The current passenger platforms are 33 cm high from the upper surface of the rail and next to the No. 2, 3, 4a, 4b and 5 railway tracks.

The No. 1a and 1b railway tracks are dead-end tracks for loading. Exiting the waiting room the railway track No. 2 can be found. Other platforms can be approached via underpass. Railway track No. 4b is for only departure.

1.4.1. Passenger trains travelling here

Passenger transport is performed by InterCity (IC) trains (8 and 16 railway lines), domestic express trains (8 and 16 railway lines) and stopping trains (8, 14 and 16 railway lines).

It carries passenger *transit traffic*:

- Budapest-Keleti railway station – Győr – Sopron and vice versa /IC/;
- Budapest-Keleti railway station – Szombathely and vice versa /IC/;
- Szentgotthárd – Szombathely – Budapest-Keleti railway station and vice versa /IC/;
- Győr – Sopron and vice versa /stopping train/;
- (- Rajka) – Hegyeshalom – Szombathely and vice versa /stopping train/.

Provides direct *domestic express train* service:

- Sopron – Győr – Budapest-Keleti railway station and vice versa /express train/;
- Szombathely – Budapest-Keleti railway station and vice versa /express train/.

Passenger train service:
- (Rajka) – Hegyeshalom – Csorna and vice versa /stopping train/;

- Csorna – Szombathely and vice versa /stopping train/;
- Csorna – Szany-Rábaszentandrás, and Csorna – Pápa and vice versa /stopping train/.

1.4.2. Freight trains travelling here

Csorna's railway station transacts transit freight transport among others:

- Sopron – Győr – Budapest and vice versa;
- Szombathely – Hegyeshalom – Rajka and vice versa;
- from own and foreign railway taking over freight trains, handing over incoming freight trains in one direction, assembling departing freight trains in one directions, and storing the stationary mixture for the frequented trains.

In Csorna the freight train traffic is significant, with approximately 100 wagons depart and arrive at the railway station daily. In the evenings and in the mornings freight train traffic, while in daytime it is more likely that passenger train traffic is the standard.

1.5. Track connections and current state

The railway tracks can be seen in the distortion site plan below from the current state (Fig. 3):

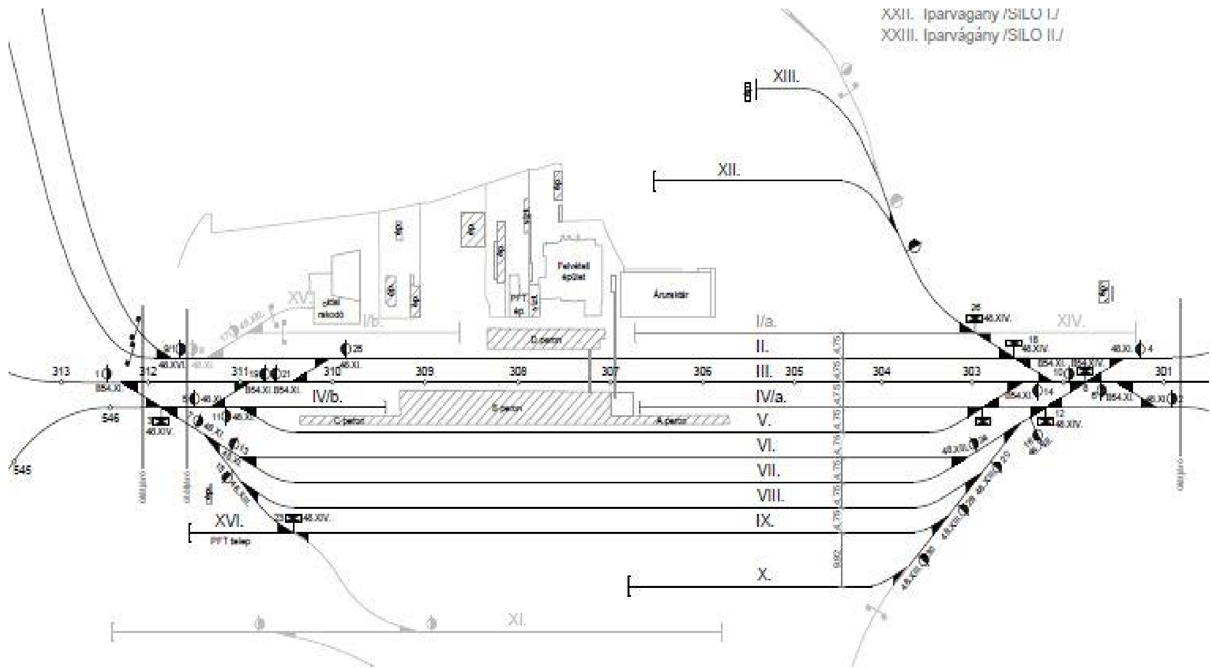

Fig. 3: Current state from track geometry of Csorna's railway station

During my on-site visit, I received a lot of information that I could not determine from operation plan. In Fig. 3 can be seen some grey lines, these are the facilities and tracks no longer in use. Besides that there are several facilities need to be renovated in the area (it can be seen in Fig. 4 and 5).

 

Figs. 4 and 5: Rarely used tracks and buildings

There are grade crossings at both entrances of the railway station (see in Fig. 6 and 7). There are two to the south-west of the passenger building and one to the north-east. One of the south-west crossings is still in use, which recently crosses the rebuilt No. 86 main road and one which was temporarily used for maintenance purposes. Currently out of service.

 

Figs. 6 and 7: The grade crossings

Fig. 8: The narrow platform A

The station has four passenger platforms as shown in the distortion site plan. From the passenger building the first platform is Platform D. The underpass on the eastern side of the building give access to Platforms A, B and C, which are essentially a larger „hat" shape together. Platform A serves passengers on the Pápa – Csorna railway line, while the track next to the platform C usually has a waiting function for storaged trains. Unfortunately, none of the platforms meet current national and EU regulations, as they are only about 1.5 meter wide (see it in Fig. 8).

During the development of the station, the inadequacies need to be corrected and if it's necessary need to be built widened and extended platforms.

The existing passenger building underwent a major renovation a few years ago, but the condition is moderate. The station has a refurbished, heated waiting room and ticket office, but accessibility is not guaranteed.

In front of the building the bike storage places are always full and there are plenty of unused buildings.

1.6. Examining the timetable and problems

I investigated the capacity of the railway station based on the operating plan and timetable (railway) by the internet sources (http://www.elvira.mav-start.hu/).

According to this, the carried traffic reaches the capacity limit. 5-5 tracks are applied for transacting the freight traffic and passenger traffic too, and that is the necessary use to keep to the timetable. I created a capacity curve based on time and number of the trains (can be seen in Fig. 9).

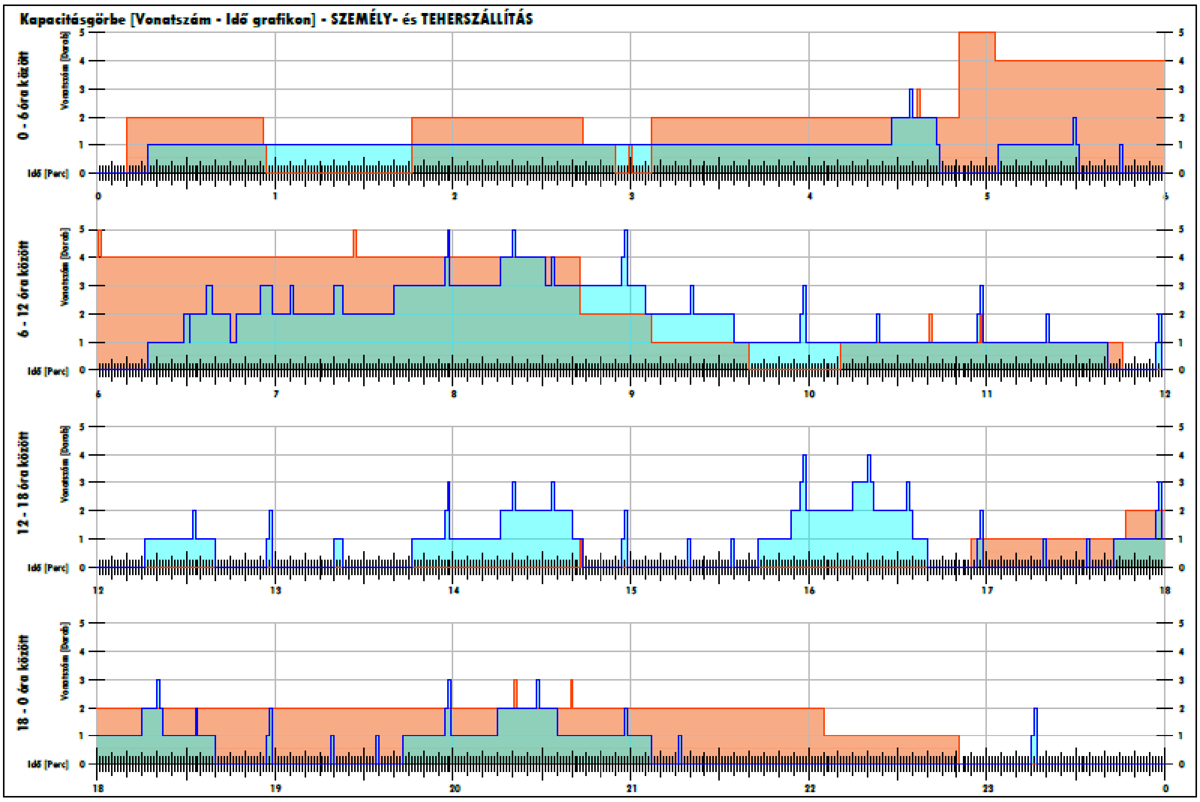

Fig. 9: Capacity curve (blue – passenger trains, orange – freight trains)

I found that it is necessary to build a new platform. Unfortunately, current timetable has no spare time, so in the event of a malfunction the system crashes and significant delays are expected. It is also important to describe that this requirement is temporary. Usually, the current station with it's tracks will perfectly meet current needs. However, even with professional traffic management, they could not solve force majeure situations, so it is important to redesign the station.

2. SWOT ANALYSIS

I considered it is important to analyze the current state and to take into account the disadvantages and advantages of the subject in several aspects.
I collected the main points of the station:

- S – Strengths;
- W – Weaknesses;
- O – Opportunities;
- T – Threats.

These are shown in the table below (Tab. 1).

Tab. 1: SWOT analysis

STRENGTHS	OPPORTUNITIES
✔ Separated freight and passenger traffic ✔ Known data, estimable development opportunities	✔ Useable neighbouring areas ✔ Flat environment ✔ Good, easy developing track geometry
WEAKNESSES	**THREATS**
✔ Road crossings at both ends of the station head ✔ Complicated switching zones ✔ Outdated railway track ✔ The proximity of the railway geometry to the station ✔ The platforms are too narrow ✔ There are not the required useable lengths on any track	✔ Heavy freight and passenger traffic ✔ Scheduled spider and central traffic control station, so great attention should be paid to the redesign of the station ✔ Railway station with small area ✔ Risk of business loss

3. DESCRIPTION OF THE VARIANTS

In each variant I tried to find the best solutions taking into account different main aspects and objectives. I present the planned variants on the basis of well-thought-out aspects. They make it easy to compare the three different solutions.

3.1. Description of the first version

During the planning of this version I created a railway station that works well from a functional point of view and does not accommodate the use of space. The switching zones have significantly simplified and every railway line is accessible from any railway line.

In addition to equal opportunities, passenger comfort and expropriations have been given a high priority.

The design can be seen in the Fig. 10 below.

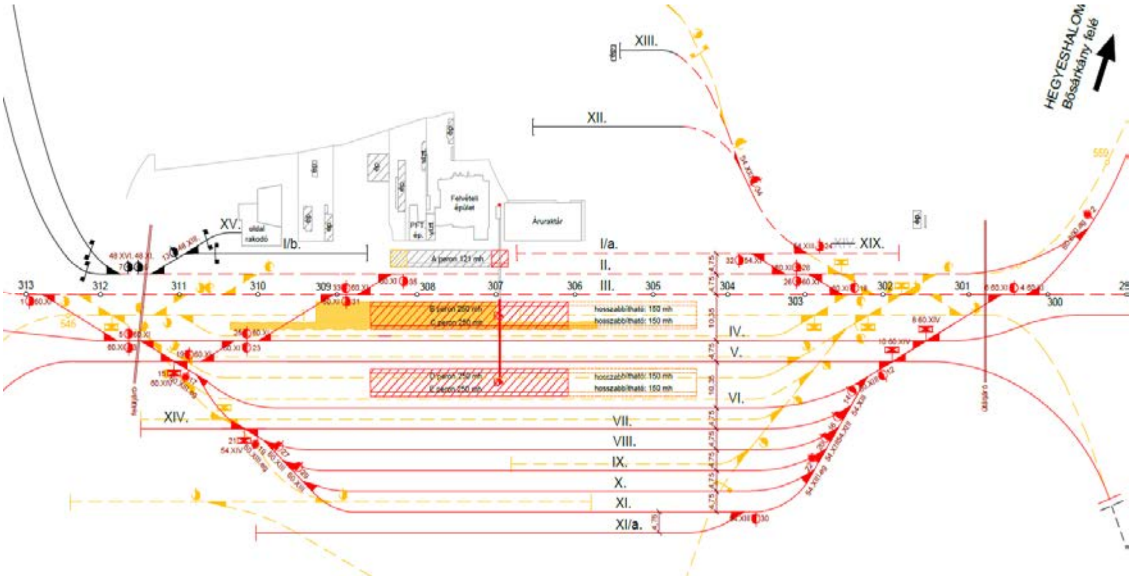

Fig. 10: The first variant of the redesign

3.2. Description of the second version

During the planning of this version I tried to create as little intervention as possible. My goal was to keep the level crossings at the station heads and the main geometry of the passing lines, the railway track axes. In addition to equal opportunities, passenger comfort and expropriations have been given also a high priority.

The design can be seen in the Fig. 11 below.

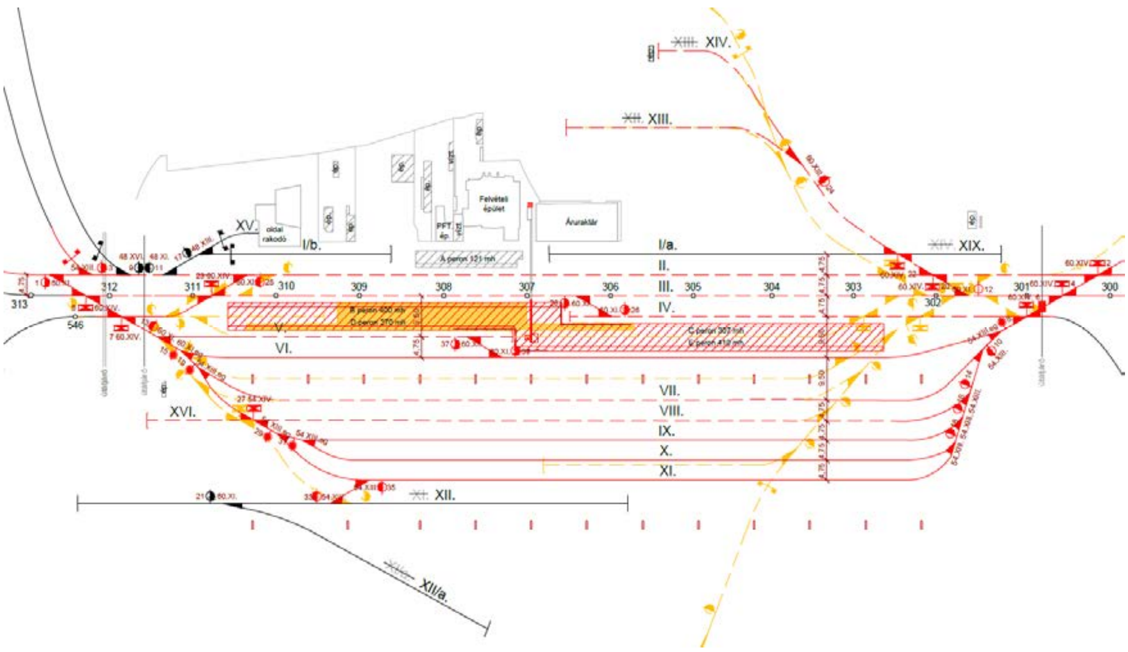

Fig. 11: The second variant of the redesign

3.3. Description of the third version

During the planning of this version I tried to develop a functional station with capacity reserve and moderate use of the surrounding areas. In addition to equal opportunities, passenger comfort and expropriations have been given also a high priority.

The design can be seen in the Fig. 12 below.

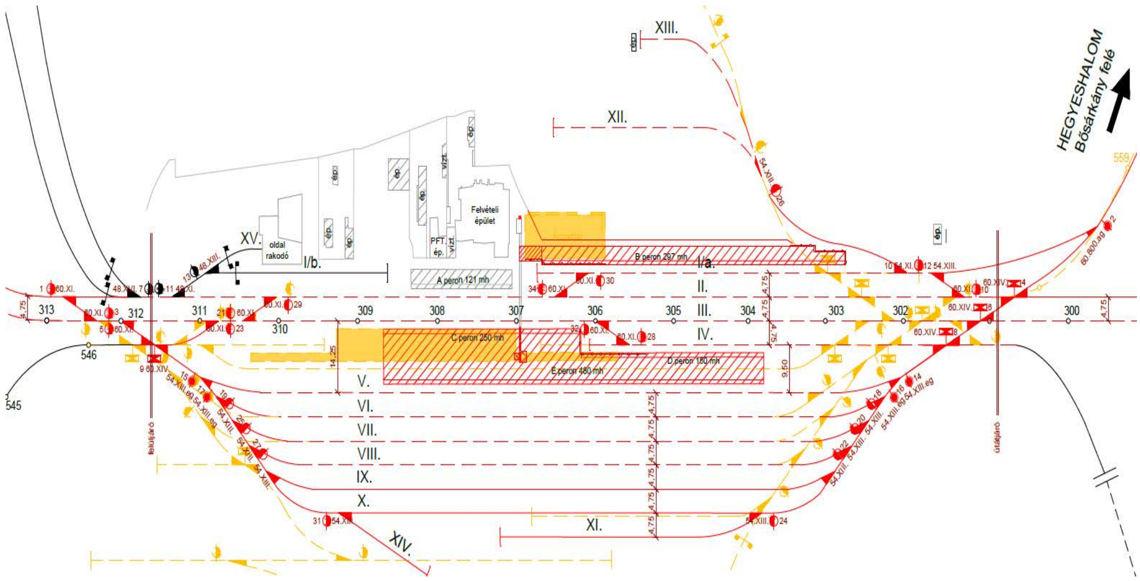

Fig. 12: The third variant of the redesign

4. EVALUATION OF DECISION VARIANTS

I performed the three variants by weighted evaluation according to the same criteria to choose the best version. Below I have collected the considerations that have been used to evaluate the sketch plans.

Hefting viewpoints:
S1 Layout of the line;
S2 Investment costs;
S3 Operating costs, maintenance requirements;
S4 Environmental protection;
S5 Equal opportunities, passenger comfort;
S6 Turnout connections;
S7 Expropriations, demolitions;
S8 Length and area of the platforms;
S9 Usable length of tracks.

Definition of weights:
The weights were determined by making a preference matrix. With the help of the matrix the achieved preference points can be summarized, based on which I was able to determine the actual value of the weights by a simple percentage calculation.

The preference matrix can be seen in Tab. 2, the value of the weights can be seen in Tab 3.

Tab. 2: The preference matrix

Viewpoints	S1	S2	S3	S4	S5	S6	S7	S8	S9	Preference
S1	-	1	0	0	0	1	1	0	0	3
S2	0	-	0	0	0	0	1	0	0	1
S3	1	1	-	0	1	1	0	1	1	6
S4	1	1	1	-	0	0	1	0	0	4
S5	1	1	0	1	-	0	1	1	0	5
S6	0	1	0	1	1	-	1	1	1	6
S7	0	0	1	0	0	0	-	1	1	3
S8	1	1	0	1	0	0	0	-	1	4
S9	1	1	0	1	1	0	0	0	-	4
Dispreference	5	7	2	4	3	2	5	4	4	-

Tab. 3: The value of the weights

Sequence	Preference	Reached number	Weighting
1.	S3	6	17
	S6	6	17
2.	S5	5	14
3.	S4	4	11
	S8	4	11
	S9	4	11
4.	S1	3	8
	S7	3	8
5.	S2	1	3

Naturally the cost estimates were also calculated with many others in the Master's thesis. The evaluation results can be seen below (Tab. 4 and 5).

Tab. 4: Evaluation

Viewpoints		Version #1		Version #2		Version #3	
S1	Layout of the line	good	4	excellent	5	medium	3
S2	Investment costs	+++	3	+	5	++	4
S3	Operating costs, maintenance requirements (number of diamond crossings with slips and turnouts on the curve)	7	5	15	2	9	4
S4	Environmental protection	good	4	good	4	good	4
S5	Equal opportunities, passenger comfort (walking distance)	200	4	300	4	180	5
S6	Turnout connections (ratio of simple and complicated turnouts)	20.6%	5	57.7%	3	28.1%	4
S7	Expropriations, demolitions	+++	3	+	5	++	4
S8	Length and area of the platforms (ratio of area to length)	27.12	4	35.17	5	18.82	3
S9	Usable length of tracks (total available track length)	9287 3156	3	9422 3827	5	7518 3343	4

Based on these considerations and the weighted evaluation, I decided to execute Version 1 in the development of the authorization plans.

Tab. 5: Evaluation results

Sign	Evaluation criterion	Weighting	Qualification		
			Version #1	Version #2	Version #3
S1	Layout of the line	0.08	4	5	3
S2	Investment costs	0.03	3	5	4
S3	Operating costs, maintenance requirements	0.17	5	2	4
S4	Environmental protection	0.11	4	4	4
S5	Equal opportunities, passenger comfort	0.14	4	4	5
S6	Turnout connections	0.17	5	3	4
S7	Expropriations, demolitions	0.08	3	5	4
S8	Length and area of the platforms	0.11	4	5	3
S9	Usable length of tracks	0.11	3	5	4
	All in all:	**1.00**	**4.12**	**3.90**	**3.95**

With the result in mind, a detailed authorization plan package has been performed for the first version.

5. CONCLUSIONS

Due to the geographical position of Csorna in the central part of Győr-Moson-Sopron county, it plays a very important role in railway transportation. The redesign of the town's railway station is a complex task that has to be examined from several viewpoints. Despite of the station's minimal size it carries significant traffic of passenger and freight traffic.

The station is affected by three railway line, the Győr – Sopron – Ebenfurt main railway line, the Hegyeshalom – Porpác railway line and the Csorna – Pápa railway line, so it is necessary to pay attention for the possible changes.

During the preparation of my thesis I analysed the status, possibilities, infrastructure, condition of the area, as well as based on the timetable and transport capacity analysis three variants were prepared. Each version would be a proper and suitable solution.

I collected the typical weaknesses, strengths, opportunities and threats for the current state as a SWOT analysis that summarizes the issues to be solved. During the redesigning I have taken into account the national and international regulations and standards in force and have tried to find the best possible solutions for railway and road areas. The requirements for accessibility and passenger comfort received special attention.

I comprehensively analysed my versions on basis of given criteria, estimate of situation and preference matrix. Finally I was able to choose the version proved to be the best.

The optimum version has been developed as a study plan as evidenced by a more detailed technical description and the planned drawing parts.

Wide, long, barrier-free platforms have been planned as well as preventive track. Each railway line received a separate railway track for it's passing and stopping vehicles.

Future planning approvals are likely to require changes to current timetables and operating plans, but a professional prepared railway organization can significantly increase the capacity of the railway station and allow even more transit traffic, possibly due to the double-railing of the No. 8 railway line.

6. LIST OF ACKNOWLEDGEMENTS

I would like to say a special thank to everyone who has contributed in some way to the completion of my thesis.

7. REFERENCES

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COMPREHENSIVE ANALYSIS TO REDUCE THE SEVERITY OF MOTORCYCLE ACCIDENTS

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SUMMARY

As a traffic engineer student, I have chosen accident prevention as my research area, and I have also studied the status of motorcyclists in Hungary, and I would like to continue this work. This research process had two important stages: the bachelor and master thesis. Current article is a brief summary of my thesis. Without completeness it presents and supports with measurements the underdevelopment of the Hungarian education, the deficiencies of the infrastructure and provides suggestions for development based on Hungarian and foreign progressive examples and lays the foundations of an application that has been completed since. In my work, I present one of the smallest groups of road users, but I think that the findings of the dissertation can be applied to some road users with some refinement to reduce the number of road accidents soon.

1. INTRODUCTION

In Hungary, an average of 21,000 people per year were directly affected by personal injury accidents each year. This means that there has been some injury to a driver or a passenger as a result of an accident. Of this, an average of 600 people died, and 5,500 people were severely injured, 15,000 more easily. Let's look behind the numbers, what do these mean in reality?

Every year around six hundred people were told that their beloved ones never return home again. Hundreds of people are struggling in everyday life to support their seriously injured family member: financially, physically and spiritually. Every year, thousands of family's life change in a negative direction. And this number is repeated every year, as the number of people directly involved in accidents is over 20,000 every year, and this number does not show a clear decrease (Hungarian Central Statistical Office, 2016).

In my thesis, I have expansively presented the tools that would be available to significantly reduce personal-injury-accidents. Though the topic of the dissertation I examine the situation of "unlimited" category of the motor bikers and it generally examines topics that can be extended to the whole road transport.

However, it is not only individuals who are affected negatively by a serious or fatal accident, but there is also a serious contribution to every tragedy at national economy level. The "price" of human life according to the European Union is one million Euros. Starting from this, we threw nearly two thousand billion forints every year on the window - if we only investigate the fatal accidents. Over the last 10 years, more than eight thousand people have lost their lives on the roads - while we are fighting for a decline in the country's population.

In my bachelor thesis I have already discussed the history of motorcycling, the difficulties of transport by motorbike from vehicle handling to infrastructure and the typical dangerous situations. In a questionnaire survey I investigated the behaviour of bikers and I examine accidents in case studies.

In this work, I would like to draw attention to improving the quality and accessibility of education and infrastructure, and to show how incidents can be mitigated cost-effectively. Although we can see more and more passive and active protection systems on the motorbikes but remember that the Hungarian motorcycle stock is getting young so slow - it is still around 15 years old. And a fifteen-year-old bike is start with carburettor choke, and some Hungarian bikers still wear the “Kiskőrösi” helmet, that is why a futuristic dream that we will use self-driving vehicles tomorrow.

2. INFRASTRUCTURE

During my work I examined two roads beloved by bikers: No. 2505. road (“Eger highway”) and road number 82. (“Csesznek road”). Based on the analysis of the accident data, it can be concluded that the primary cause of the accidents is the inadequate speed selection of road conditions (road tracing, road quality and condition).

In this context, I would like to mention the classic sentence on infrastructure, which states that: “The vehicle must be driven in accordance with traffic, weather and visibility and road conditions (road tracing, road quality and condition); attention must be paid to the characteristics of the vehicle, the passengers and the load” (KPM-BM, 1975).

This is one of the most quoted passages of the “Rules of the Road”, yet the least understood. It does not draw any boundaries, does not give an explanation, does not give help, only states: drive like this. And the drivers are trying to travel with all the knowledge they have.

But how can we prepare for those above? We'll take this multi-sentence for its elements:

- *Traffic conditions*
  The driver has learned to drive in “small-town” conditions. Experienced junctions directed by traffic light, differently provided railroad crossings, roundabouts, but for example, have not travelled many times on a parallel traffic road. He is travelling to Budapest with a fresh license, as nothing prohibits him. How does he need to travel in a fast-paced, difficult-to-use capital city? Where did he practice this?
- *Weather and visibility conditions*
  If someone started to learn to drive in the spring, his driver's license is in his hands in autumn. When in a rainy, slippery time the vehicle slipping down, what to do in such a situation? When did you acquire it?
- *Road conditions*
  There is an oil spill in front of the driver outside the populated area at night. If he even notices when he was practicing the sudden avoidance? Does he know what to do with a sudden sliding vehicle?
- *The characteristics of the vehicle*
  An experienced, older motorbiker loan gets a new type of sports engine. He tries to stop the rear brake, the biker and the bike are crashing, because only his rear brakes were enough on his motorbikes. Looking for his memories when he was told that the first brake on modern engines is more effective?

- *Passenger (load)*
  A young man and his girlfriend are going to ride a bike. The man never carried a passenger, what must he say to the lady, how to prep her for the trip? What does he expect the bike to do with the extra weight? In the bend the bike obeying the laws of physics, but the lady dolly in her fears, the bike straightens, the pair do in the ditch. They didn't drive fast, they were driving regularly, but now they are expecting the ambulance. Why is that?

The cases above are daily and accidental situations for which education is not or only partially prepared the drivers, but the law still requires the mentioned knowledge.

In addition, improving the quality of domestic roads is an ongoing issue. It would be important to have a prominent role already in the planning phase:

- road safety aspects,
- protection of unprotected road users,
- the self-explaining and forgiving nature of the road,
- transparency, awareness-raising and explanatory role of signalling systems;
- investigation of modern road construction and traffic engineering (e.g. Vision 0).

All of this are true when the contractor side works in the best quality and the maintenance is continuous.

3. EDUCATION

95-98% of road traffic accidents are caused by human error, as a result to reduce the accidents the elimination or improvement of the human factor is necessary. The massive appearance of autonomous vehicles is still awaiting, so developing the education is an obvious solution. For this reason, I have examined the foreign education systems to be followed and the Hungarian driving courses, because of these, it can be said that Hungarian education is not lagging in terms of compulsory hours, and even more! Practical hours are far from the European average. But as the British example shows (where there is no required number of hours), it is not necessary to regulate it, as without the number of hours there the learner's skills determine the length of the training. However - despite our interviews with motorcyclists - they were only on a fraction of the required hours. There were also bikers who only saw his instructor three times - included the exams. But if they took part in every hour, there was no more than just unfocused circles, that are not guided by usable direction. So, there is a problem with quality rather than quantity.

In the following, I have collected the best practices of the examples examined, which included in Hungarian education would significantly increase its quality:

- competency test,
- recognition of latent hazards,
- vehicle features, motorcycle types,
- complementing education with an application,
- vision,
- probation,
- biphasic education,
- instructor’s opinion about the student,

- evaluation of trainers and schools
- driving on highways, in the dark, at dusk
- routine training based on driving trainings,
- post-training.

The United States' Motorcycle Safety Foundation (MSF) is also implementing an educational program that like Hungary's safety driving centres program. It is a perfect summary of where Hungarian basic motorbike training should go as soon as possible. In off-road areas, typically in larger parking lots, routine instruction is provided, where students learn throttle, shifting, braking, straight-line driving, cornering, and then gradually building turns and emergency braking. In order to prove the importance of the following, I performed measurements with the help of drive-training instructors (Róbert Kovács-Birkás - Hungaroring Motorbike Academy, Attila Urbán - Safety Hungary) and an accelerometer (Sensor Kinetics Pro).

1. Motorcycle Familiarization
   - Review T-CLOCS pre-ride inspection
   - Identify location and operation of important controls and major parts
   - Review mounting/dismounting procedures
   - Review elements of good posture
2. Using the Friction Zone
   - Become skilled in using the clutch friction zone for control
3. Starting & Stopping Drill
   - Coordinate the friction zone, throttle, and brakes to control the motorcycle
   - Start out and stop with precision and control
4. Shifting & Stopping
   - Shift gears and stop smoothly
5. Basic Skill Practice
   - Refine low-speed manoeuvring skills
   - Refine throttle use and brake manipulation for corners
6. Pressing to Initiate and Adjust Lean
   - Understand the manoeuvring elements needed for negotiating curves
   - Experience the effects of handgrip pressure and handlebar movement to initiate and adjust lean
7. Stopping More Quickly & Tight Turns from a Stop
   - Develop a feel for progressive braking pressure to stop more quickly without skidding
   - Practice making a sharp turn from a stop
8. Stopping Distance Demonstration
   - Observe a demonstration of the reaction/braking parts of total stopping distance
   - To understand effects of speed on braking distance
   - To relate the results to intersection strategies
9. Limited-Space Manoeuvres
   - Refine manoeuvring skills to allow turns in limited spaces
   - Learn the counterweighting technique
10. Stopping in a Curve
    - Learn to maintain control while stopping in a curve
    - Understand traction management

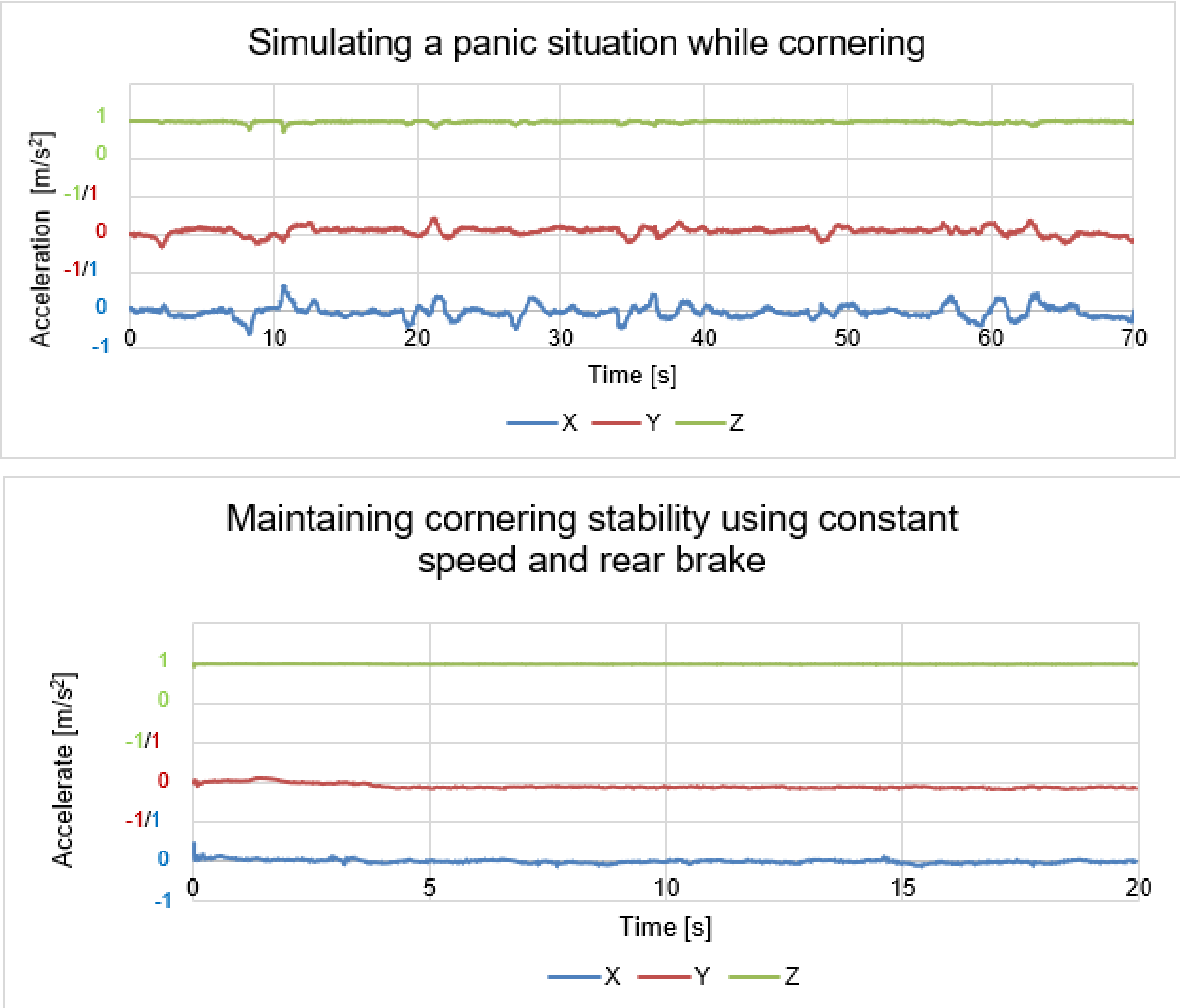
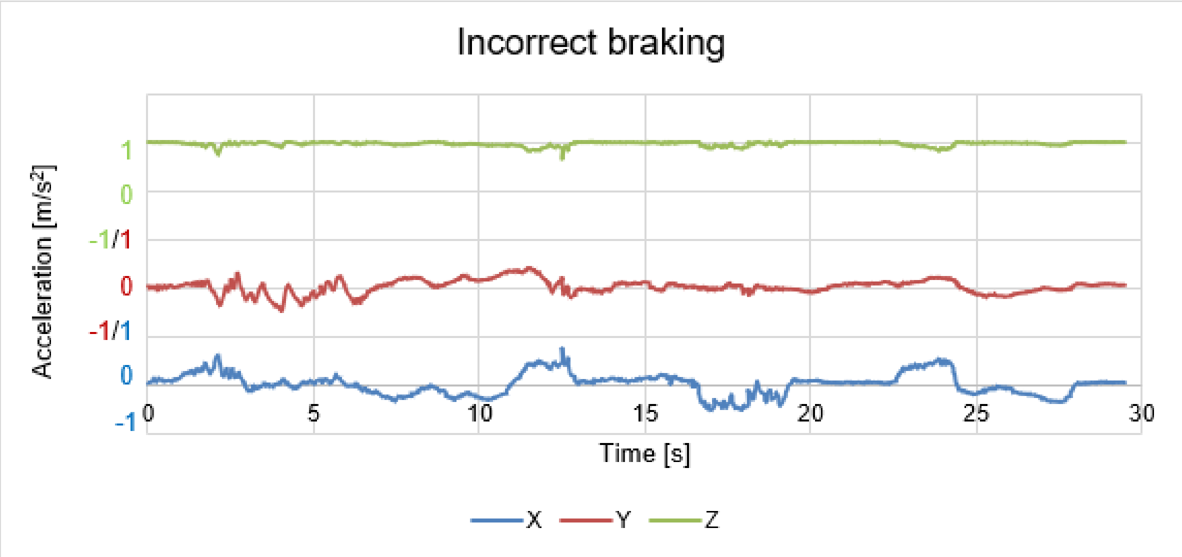


Fig. 1: Measurements show a significant increase in cornering stability when the rider handles the controls correctly and is aware of the correct seat position

11. Curve Judgment
    Cornering - unlike driving - requires much more attention and special techniques on the motorbike. This is illustrated in Fig. 2.
    - Improve skills for negotiating multiple curves
    - Understand the "search-setup-smooth" strategy:
        - search: curve survey,
        - setup: finding, finding, right speed, technique, curve, posture, gaze,
        - smooth throttle control, continuous, continuous monitoring of curve and exit points (keep an eye out for a single point), and fine, no sudden brake application when needed.
12. Multiple Curves & Lane Changes
    - Practice negotiating curves and lane changes
    - Understand safety margins and gap selection
13. Crossing an Obstacle & Swerving
    - Learn techniques for crossing over obstacles
    - Execute a basic swerve
    - Practice turning from a stop
    - Refine slow-speed weaves



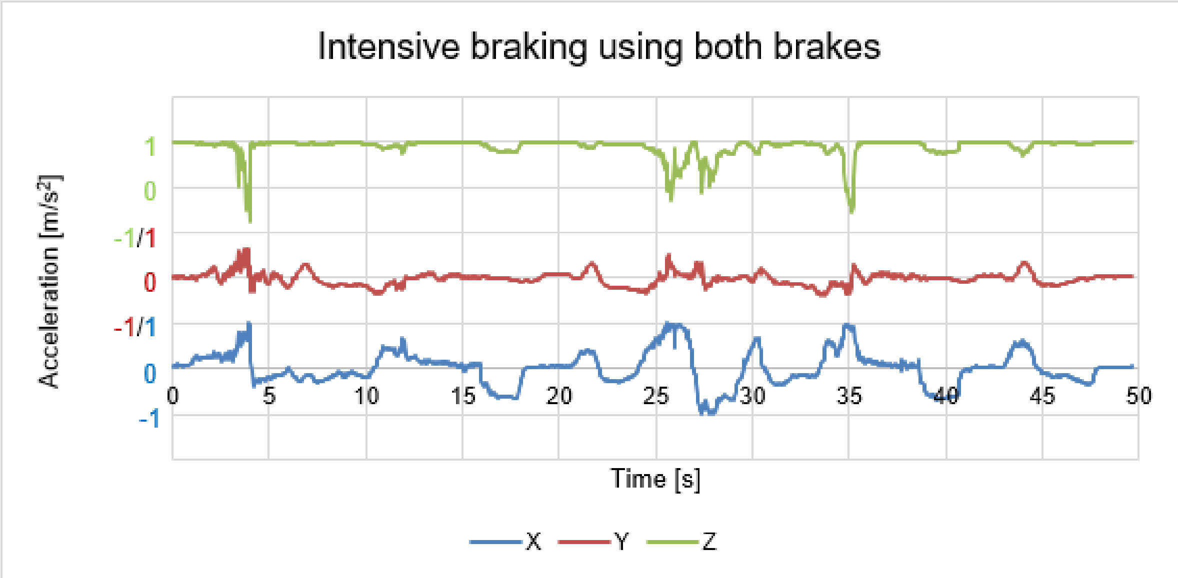


Fig. 2: The diagram of properly executed progressive braking clearly shows when the braking took place, while " ad hoc " braking shows a more elongated, less definitive braking. Of course, this can also be measured over braking distances.

14. Skill Practice
    - Capstone exercise that combines a variety of manoeuvres

+ Skill Test
    - To assess basic skills using a cone weave, normal stop, turning from a stop, U-turn, quick stop, obstacle swerve, and cornering manoeuvre.
    - To demonstrate basic motorcycle control skills and ability to avoid an obstacle
    - To demonstrate ability to use the proper technique to negotiate a curve (Motorcycle Safety Foundation, 2018)

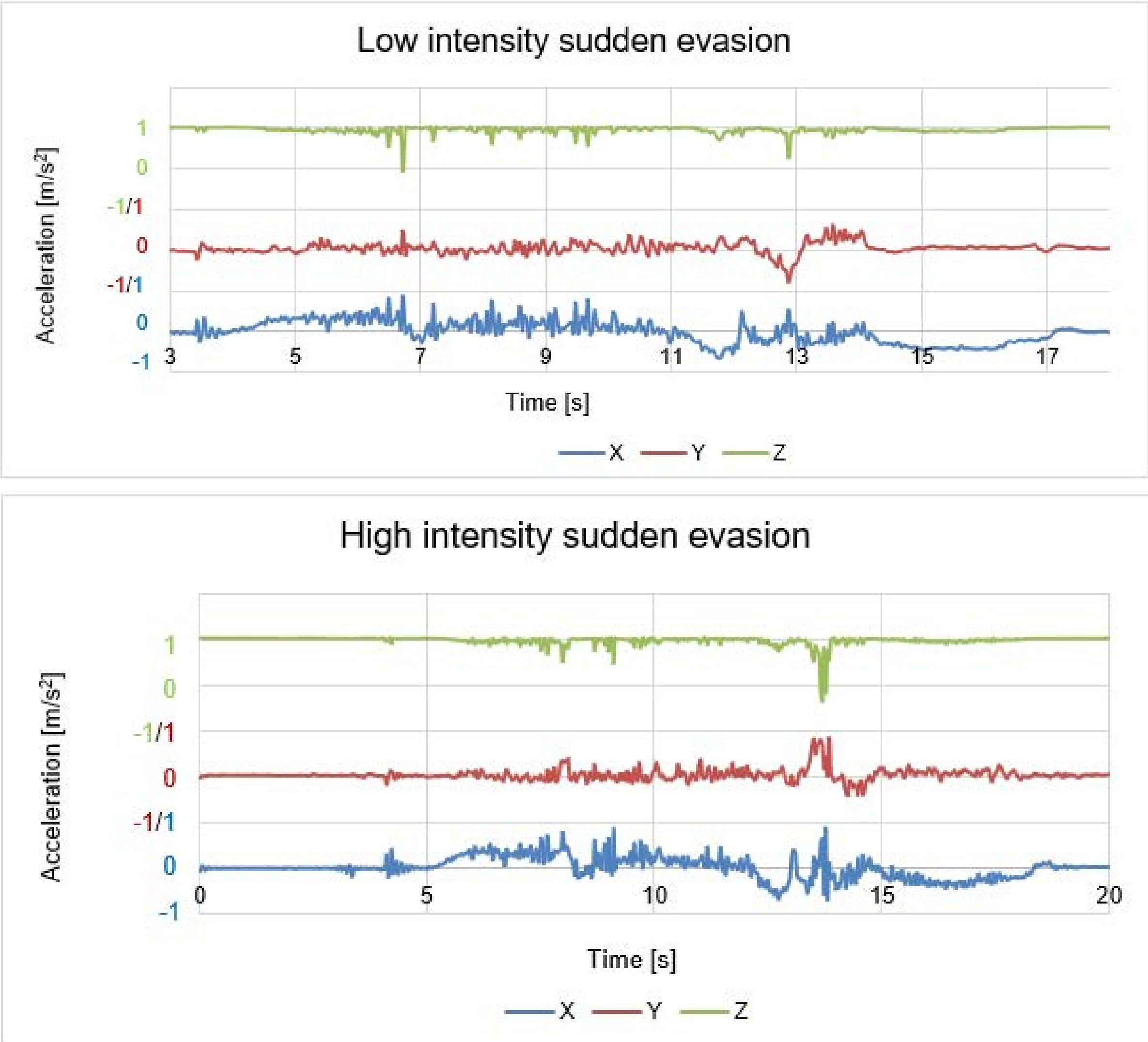


Fig. 3: Quick and accurate action is required to avoid an unexpected obstacle. This requires attention, proper seating position, effective emergency braking, gaze guidance, countersteering, avoidance

The above is necessary to prove that the student has the basic skills to control the motorcycle, to avoid obstacles, and to use the proper technique when cornering and braking.

The course concludes a test and a practical assessment. There is little chance that basic education will be transformed in Hungary soon but driving training can already provide this level in Hungary, but this will cause additional costs for riders.

Depending on the school, the cost of basic education is typically tens of thousands of forints, which is costly expense for many.

On the other side of the material, there is the material and moral esteem of instructors what is important to make careers attractive to talented, determined and capable people.

However, the cost of an accident to the economy is in any case greater than the cost of a driving-training course. According to the website of Continental Safety Hungary, besides the current number of students, the cost of a two-day training course is about HUF 92,000, half of which is paid by the sponsors; thus, the students pay HUF 49,000. The "cost" of a minor injury accident is HUF 2,520,918, a major injury accident is HUF 34,589,345, and the accident death is estimated at HUF 257,954,438 in 2017 (FŐMTERV Zrt., 2017).

If we look only at accidents caused by motorcyclists, almost all of which are due to a lack of driving or thinking, then the chart below shows that one third of all accidents could cover the cost of at least one basic training for all motorists.

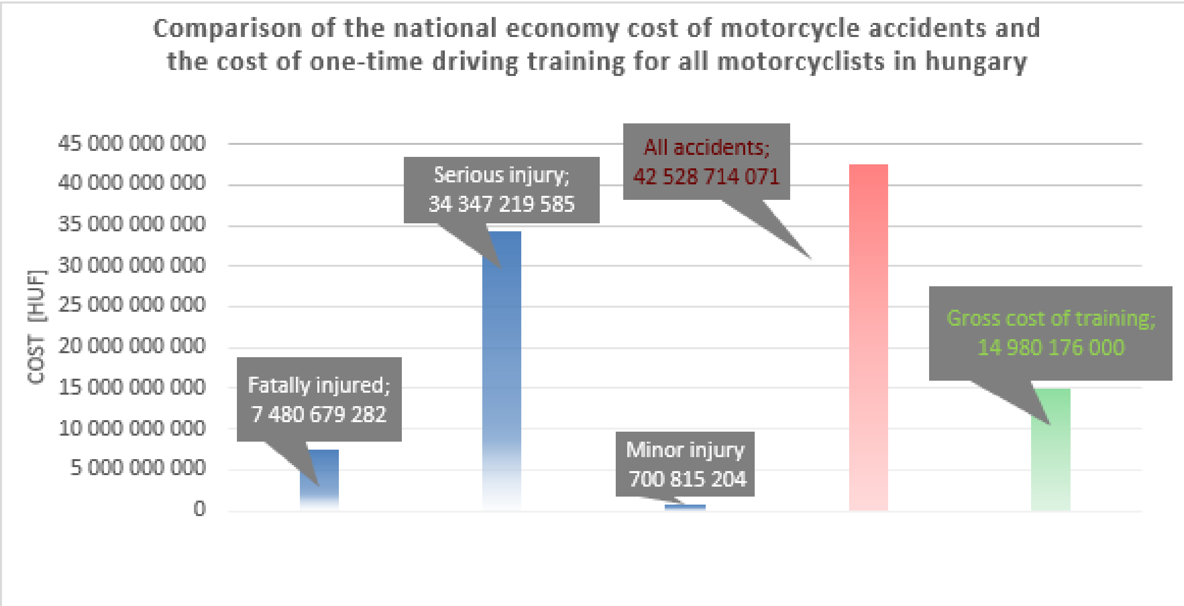


Fig. 4: Comparison of the national economy cost of motorcycle accidents and the cost of one-time driving training for all motorcyclists in hungary

Anyone who has attended a training once understands the importance of updating the knowledge at least once at the start of each season.

Thus, a one-off action of this kind could lay the foundations for a more responsible biker society.

4. BIKECALL

In the case of motorcyclist’s journeys on individual or non-dedicated road network usage are outstanding, in which case making an emergency call in the event of an accident may be difficult or impossible in certain injuries. It would be important for them to have an easy-to-use or possibly automatic tool that can help them in such cases. There are several serious accidents witch severity could have been reduced if the help had been received on time.

A well-known concept of emergency patient care is the so-called “gold hour”, where treatment can be most effective if it can be started by professionals within one hour of the incident. From this vital hour takes on average 10 to 15 minutes between the occurrence of an accident and its reporting, which could be eliminated by a well-designed tool.

In addition, it should be taken into a count that there are accidents that occur outside populated areas or in poorly visible, accessible locations, in which case a tool to assist in locating the injured person or signalling an accident would be justified.

And almost any lower-end mobile phone available today can meet those needs by downloading the right application. And while the eCall emergency response system would provide a solution to the above problem, its planned rollout is not yet available, or a version optimized specifically for motorcyclists is not yet available - and the application aims to provide workaround or alternate solution.

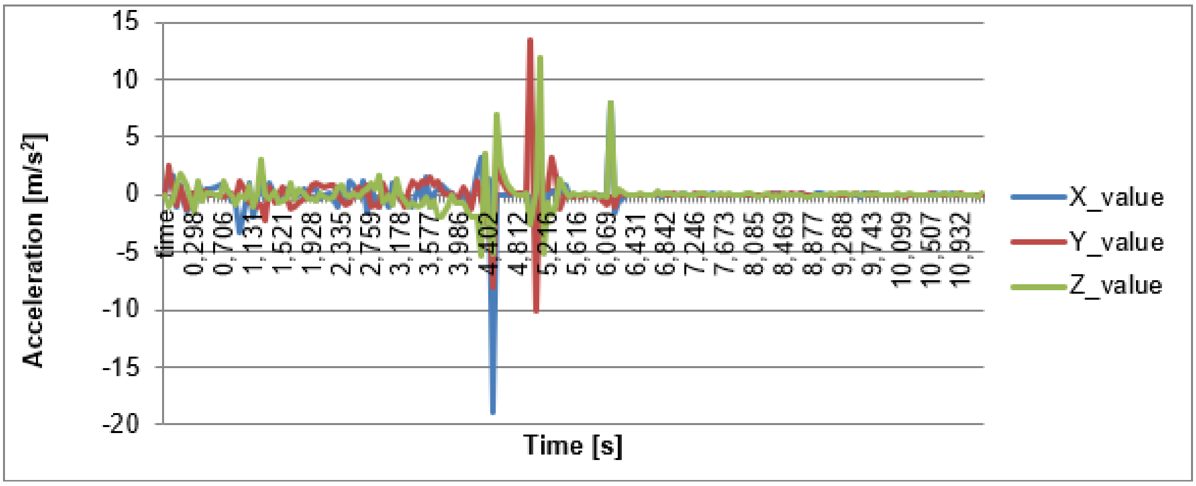


Fig. 5: Collision illustration 20 Hz, linear values

Fig. 6: Screenshots of the BikeCall

5. CONCLUSIONS

The accident data and experiences presented in the dissertation suggest that the Hungarian motorcycle traffic and education is extremely underdeveloped, although there are positive efforts. Reducing the number of fatalities and injuries requires effective measures, but this requires systemic changes.

The study focuses primarily about motorcyclists, including the large category, but most of its findings apply to all categories. The deficiencies experienced by motorcyclists can be traced to the smaller - mostly the moped category - as well as the deficiencies in the driving morale and driving technique of small and large motorists.

In my work I researched the relationship between road conditions and the human factor, which showed that while there is much to do to improve the quality of the former, the main error factor is primarily the human, but human and its capabilities can be improved.

An example of this is the high level of knowledge of the driving instructors showed in my dissertation, proven by my measurements, which can be acquired by anyone with proper practice and guidance, but the background must be created in a wide, accessible format.

During my activity I concluded that not only the number of accidents but also their severity should be reduced. For this reason, I see great potential in developing the BikeCall application so that can help as soon as possible to the road users who are in emergency situations.

Although I research the smallest number of road users, the purpose of my work is to draw attention to the anomalies of Hungarian transport and its education, not only among motorcyclists, but among the general population.

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MAÚT25 INTERNATIONAL SCIENTIFIC SYMPOSIUM

Budapest, Hotel Gellért, 17–18 September 2019

PROGRAMME

17 September 2019 (Tuesday)

9:00	**Dr. László Mosóczi** **State Secretary, Ministry for Innovation and Technology**	**Welcome Speech**

Session 1 – Opening
Session Chair: András Mayer

9:15	Szabolcs Nyiri Chairman, MAÚT	Opening
9:30	Francois Chaignon	Future of HMA: What are the needs?
10:00	Dr. Ferenc Horvát	Utilisation of research and development results on professional field of railway infrastructure
10:15	Dr. Michel Virlogeux	Long span bridges

10:30 Coffee break

Session 2
Session Chair: Zsolt Thoroczkay/Dr. Michael Rohleder

11:00	Lukas Prettner	Cost saving potentials by the use of new rail steels based on life cycle cost considerations
11:30	Dr. Akio Kasuga	Evolution of bridge construction – Non-metallic bridges
12:00	Dr. Kamil Elias Kaloush	Sustainable pavements: Bridging research and industry practices
12:30	Dr. Lajos Kisgyörgy	From research to innovation: Case study of a road pavement research project

13:00 Lunch break

Session 3
Session Chair: Zsolt Völgyesi/Zbigniew Kotlarek

14:00	Adrián Horváth	I'm innovating, therefore I am – (Everyday) practice of innovation
14:30	Heinz Ossberger, Albert Jörg	Modern highspeed turnout system solution – From geometric and structural requirements to signaling integration
15:00	Dr. Igor Ruttmar	Perpetual pavement design, S8 Expressway
15:30	János Béli	Rail diagnostic developments
15:45	Erika Juhász, Dr. Szabolcs Fischer	Analysis of crushed ballast particles under laboratory measurement conditions

16:00 Coffee break

Session 4
Session Chair: József Attila Szilvai/Ján Šedivý

16:30	Ede Andráskay	Gotthard: from the path to the railway base tunnel – 2000 years of the Alpine crossing
17:00	Dr. János Szendefy	Possibilities to determine the bearing capacity of earthwork and description of methods for improving the bearing capacity
17:30	Dr. György L. Balázs	New materials for concrete bridges

18:00 Awarding Ceremony

19.00 Gala Dinner

18 September 2019 (Wednesday)

Session 5
Session Chair: Dr. György L. Balázs/Dr. Martin Fellendorf

9:00	Dr. S.K. Jason Chang	Mobility as a service: research, development and case studies in Taiwan
9:30	Peter Veit, Stefan Vonbun, Markus Heim	Elastic elements in track influencing total track costs and reducing vibrations
10:00	Zsolt Boros, Juraj Soták, Filip Buček, Maroš Halaj, Zsolt Benkó	Homogenisation layer – A green and innovative answer for reconstruction of old cement-concrete roads

10:30 Coffee break

Session 6
Session Chair: Dr. Kornél Almássy/Dr. Philippe Chifflet

11:00	Harald S. Müller, Michael Vogel	Service life design for traffic infrastructure of concrete – State of the art and new approaches
11:30	Dr. Alfred Weninger-Vycudil	European trends in pavement management – a challenge for Austrian road administrations
12:00	Dr. László Dunai, Adrián Horváth, Balázs Kövesdi	Theoretical and experimental analyses of historical Danube bridges in Budapest
12:20	Dr. Ervin Joó, Zoltán Előhegyi	Experiences of operating test of turnouts with different rail inclination and rail material installed in the same railway station
12:40	József Grabarits	The past, present and future of tunnel construction in Hungary in the light of international practice

13:00 Lunch break

Session 7 Session Chair: Dr. Ferenc Horvát/Dr. Gábor Köllő		
14:00	Gábor Szabó	Construction of the steel structures of the new Danube bridge in Komárom-Komarno
14:10	Tamás Attila Tomaschek	Towards connected and automated driving in Hungary – The changing role of the road operator
14:20	Dr. Tibor Mocsári	Road safety investigations in design and building
14:30	Dr. Zoltán Major	Special problems of embedded rails in urban tracks
14:40	Dr. Frigyes Törőcsik	Past and present of road construction technologies
14:50	Pál Pusztai	Megyeri bridge: Cable-stayed bridge on the main Danube branch
15:00	Dr. Ákos Vinkó, Evelyn Gonda, Attila Csikós	Smartphone motion sensor-based ride quality test for detecting vehicle safety and track maintenance issues on tramways
15:10	Dr. Pál Zoltán Bite	The concept of permanent noise monitoring stations along highways and railwaylines
15:20	Erika Juhász	GYSEV 8 railway line, complex development plan of Csorna's railway station
15:30	Judit S. Vígh	Comprehensive analysis to reduce the severity of motorcycle accidents
15.40	**Szabolcs Nyiri** **Chairman, MAÚT**	**Closing of MAÚT25 International Scientific Symposium**