Seismic Performance and Damage Assessment of Hungarian Road Bridges

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1. Introduction

1.1. Problem description

In high seismicity regions (such as the West Coast of the US, Japan, the Mediterranean region of Europe etc.), people are aware of significant earthquakes, lessons learnt from previous devastating seismic events induced the improvement of seismic design and also the application and evaluation of retrofit strategies. In moderate or low seismic areas (e.g. eastern part of the US, northern parts of Europe etc.), seismic risk mitigation efforts have lagged, because in these regions large earthquakes are infrequent and may not have been experienced for over a century while modern design methodologies and codes have been developed. Most bridges were designed with no seismic consideration; and due to inadequate detailing, even moderate earthquakes might cause severe damage. For this reason, the uniform European standard (Eurocode, EC8-1 [1], EC8-2 [2] and EC8-3 [3]) prescribes the seismic design of new and retrofitted structures even in moderate and low seismic regions.

With the introduction of Eurocode, seismic design became obligatory in Hungary. Per the Hungarian road bridge standard (ÚT [4]), only bridges with spans over 50 m had to be designed for seismic actions; thus several bridges were built without seismic design. In accordance with the EC standard, a seismic zone map was released in 2006 [5] showing that the design peak ground acceleration (PGA) is in the range of 0.08-0.15 g. This was a significant increase compared to previous values. The moderate seismicity and the lack of seismic design raise the question how the bridges behave in case of a seismic event. Experience with existing and new structures in the last decade [6],[7],[S10] showed that a large portion of road bridges may be vulnerable to earthquake loads. Therefore, it is an urgent and important issue to evaluate the seismic performance of road bridges in Hungary.

1.2. Methodology and primary tasks

State of the art seismic vulnerability assessment is based on analytical fragility curves. Fragility curves are conditional probability statements giving the probability of reaching a particular limit state (LS) for a given intensity measure (IM) level. For the unconditional probability of failure, the IM exceedance rate for the reference period is needed, which is provided by the seismic hazard curve of the site [S8].

![Fig. 1 Illustration of the probability of failure.](image)

Fragility curves can be used efficiently in both pre- and post-earthquake situations. Comparing the seismic performance via fragility curves, vulnerable bridge types and configurations can also be highlighted, retrofit prioritization can be made. They can also be used to assign a level of functionality to each bridge after a seismic event that is essential for the determination of emergency routes and recovery planning.
The long-term goal of the research is to carry out a nationwide seismic performance evaluation of road bridges in Hungary. A similar regional study was conducted in the eastern part of the US [8], Italy [9], Turkey [10], however but to the best of my knowledge, there is no such regional study in a moderate seismic zone in Europe. The developed evaluation framework is presented in Fig. 2; 5 different modules are worked out in detail, and the evaluation is automatized by creating connection between these modules [S7]. The nationwide evaluation of thousands of structures can be carried out automatically if essential data of each bridge is available.

2. Determination of the seismic load

A probabilistic seismic hazard analysis [11] is employed for Hungarian circumstances [S16] with which the hazard curves (required to calculate the probability of failure; Fig. 3a) and the site-specific spectra (Fig. 3c) for various sites are determined. Comparison between site-specific and EC8-1 standard spectra is also carried out.

According to Fig. 3b, earthquakes with small distance (<10 km) and <5.5 magnitude contribute most to the hazard at the design PGA level. In this case, the standard Type 2 spectrum should be used per EC8; which is confirmed by Fig. 3c: the Type 2 spectrum describes the site-specific spectrum better than the Type 1 spectrum. Even so, Type 1 spectral shape is proposed by the Hungarian National Annex to EC8, thus its modification is suggested [S6].

For fragility analysis, non-linear time-history analysis is needed. A record generation module is incorporated in the seismic performance evaluation framework to generate artificial ground motions for time-history analysis.
The artificial motions are generated with an iterative procedure to match a specific target spectrum [S1] (Fig. 4a). At the end of the iteration process, adjustments are made to ensure that the residual velocity and displacement are zero (Fig. 4b). The algorithm is efficient, 3 iteration steps are sufficient to achieve results with negligible error.

Fig. 4 Artificial record generation: iteration (a); correction (b).

Artificial records generally have an excessive number of cycles of strong motion and unrealistically high energy content compared to real ground motions. As a next step, a state of the art record selection procedure [12] is employed for Hungarian sites [S16]. Records are selected from thousands of ground motions; the selection is carried out in a way that the distributions of the intensity measures follow a theoretical distribution specific to the site (Fig. 5a). Such records are illustrated in Fig. 5b. The theoretical distributions can be used to characterize the expected earthquake characteristics. For instance, the median significant duration (D_{595}) is under 10 s which is in accordance with the disaggregation results (the dominant short distance ground motions can be characterized by short duration).

3. Evaluation of the road bridge database

A database of existing road bridges is created from the raw data of the Integrated Bridge Database (IBD) operated by the Hungarian Transport Administration [13]. The structure of the database and the stored data types are investigated; and the data required for a sophisticated seismic analysis and damage evaluation is determined. The database is created for road management purposes, it contains only limited information. It is not sufficient for a
detailed description of each bridges, thus bridges are grouped into classes and archetypes are chosen for further analysis. The data collection is essential to carry out a regional evaluation, therefore a three-phase extension is worked out and a possible structure of the extended database is proposed [S5].

There are about 12 000 bridges in the database (Fig. 6a), most of the bridges are short-span single span bridge. With regard to the seismic effect, multi-span bridges with longer spans (>10 m) are critical (Fig. 6b). Typically, these bridges are on primary roads (highways, autoroutes). The contribution of primary road bridges to the total value is 80%, therefore the bridge classification is carried out only for primary road bridges (~3200 bridges).

There are about 12 000 bridges in the database (Fig. 6a), most of the bridges are short-span single span bridge. With regard to the seismic effect, multi-span bridges with longer spans (>10 m) are critical (Fig. 6b). Typically, these bridges are on primary roads (highways, autoroutes). The contribution of primary road bridges to the total value is 80%, therefore the bridge classification is carried out only for primary road bridges (~3200 bridges).

![Fig. 6 Existing bridges in Hungary (red – simple span; black – multi-span bridges). a) All the bridges; b) bridges with maximum span length over 10 m.]

Table 1 Classification of primary road bridges based on structural type, relative number and value.

<table>
<thead>
<tr>
<th>Structural type</th>
<th>Number (%)</th>
<th>Value (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reinforced concrete</td>
<td>83.1</td>
<td>44.4</td>
</tr>
<tr>
<td>Precast multi-girder</td>
<td>49.9</td>
<td>32.6</td>
</tr>
<tr>
<td>Monolithic slab</td>
<td>24.1</td>
<td>5.5</td>
</tr>
<tr>
<td>Monolithic frame</td>
<td>7.6</td>
<td>0.8</td>
</tr>
<tr>
<td>Prestressed box girder</td>
<td>0.7</td>
<td>4.2</td>
</tr>
<tr>
<td>One or two-box girder</td>
<td>0.7</td>
<td>1.3</td>
</tr>
<tr>
<td>Steel</td>
<td>1.0</td>
<td>12.4</td>
</tr>
<tr>
<td>Riveted steel truss</td>
<td>0.4</td>
<td>4.7</td>
</tr>
<tr>
<td>Welded girder</td>
<td>0.4</td>
<td>3.5</td>
</tr>
<tr>
<td>Welded box with orthotropic deck</td>
<td>0.2</td>
<td>4.2</td>
</tr>
<tr>
<td>Composite</td>
<td>1.3</td>
<td>5.7</td>
</tr>
<tr>
<td>Composite girder</td>
<td>1.0</td>
<td>1.7</td>
</tr>
<tr>
<td>Composite box girder</td>
<td>0.3</td>
<td>4.0</td>
</tr>
<tr>
<td>Concrete or stone arch - tubosiders</td>
<td>9.0</td>
<td>1.2</td>
</tr>
<tr>
<td>Tubosider</td>
<td>5</td>
<td>0.9</td>
</tr>
<tr>
<td>Concrete, RC pipe</td>
<td>3</td>
<td>0.2</td>
</tr>
<tr>
<td>Stone or masonry arch</td>
<td>1</td>
<td>0.1</td>
</tr>
<tr>
<td>Special bridges (e.g. Duna bridges)</td>
<td>2.3</td>
<td>33.9</td>
</tr>
</tbody>
</table>

The next step is to classify the bridges based on their construction material and structural type. Table 1 shows that the majority (>83%) of primary road bridges are reinforced concrete bridge, moreover their approximate value is more than 40% of the total value. Steel and composite girders and also special bridges (such as large span river bridges; e.g. over the Danube river) have significant contribution to the overall value, however their number is not considerable (<5%). Special bridges are excluded from the evaluation, since they need individual analysis, and these bridges are too specific, general conclusions cannot be drawn. Other bridge types are also excluded: 1) pipes, arches and other buried structures, since they are not conventional bridges, their behavior is significantly different; 2) riveted steel truss bridges requires special analyses and their actual condition should be surveyed; 3) reinforced concrete frames are typically short single span bridges, thus they are not sensitive to earthquake loads.
Finally, 8 bridge types are selected (Table 2, Fig. 7) considering other classification aspects (e.g. monolithic or conventional bearing; seat type or integral abutment). The selected bridge types represent well typical bridges on primary roads. According to Table 2, the most commonly used structural type is the precast multi-girder bridge with a number of about 1600 bridges (~50%) followed by slab bridges (~24%), while typically the others are single span or multi-span continuous girders.

Table 2 Selected representative bridge types on primary roads.

<table>
<thead>
<tr>
<th>No.</th>
<th>Class type</th>
<th>Abbreviation</th>
<th>Bearing type</th>
<th>Typical bent type</th>
<th>Relative frequency (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Precast multi-girder</td>
<td>PMG-I</td>
<td>Monolithic</td>
<td>Multi-bent</td>
<td>45.0</td>
</tr>
<tr>
<td>2</td>
<td>Precast multi-girder</td>
<td>PMG-NI</td>
<td>Elastomeric + monolithic</td>
<td>Multi-bent</td>
<td>5.0</td>
</tr>
<tr>
<td>3</td>
<td>RC slab</td>
<td>SLAB</td>
<td>Monolithic</td>
<td>Multi-bent</td>
<td>24</td>
</tr>
<tr>
<td>4</td>
<td>RC box girder</td>
<td>RC-B</td>
<td>Conventional bearing</td>
<td>Single bent</td>
<td>1.4</td>
</tr>
<tr>
<td>5</td>
<td>Composite girder</td>
<td>COMP-I</td>
<td>Conventional bearing</td>
<td>Multi-bent</td>
<td>1</td>
</tr>
<tr>
<td>6</td>
<td>Composite box girder</td>
<td>COMP-B</td>
<td>Conventional bearing</td>
<td>Multi-bent</td>
<td>0.3</td>
</tr>
<tr>
<td>7</td>
<td>Steel girder</td>
<td>STEEL-I</td>
<td>Conventional bearing</td>
<td>Single bent</td>
<td>0.4</td>
</tr>
<tr>
<td>8</td>
<td>Steel box girder</td>
<td>STEEL-B</td>
<td>Conventional bearing</td>
<td>Single bent</td>
<td>0.2</td>
</tr>
</tbody>
</table>

Fig. 7 Typical cross-sections for each bridge class.

In the last step, statistical tools are used to determine the range of the most important structural attributes (number of spans, pier height, deck width, span length etc.) of each bridge type. Note that only PMG and SLAB bridges are sufficiently high in number to draw a reliable parametric field for the most important structural attributes. Accordingly, parametric studies are conducted only for these two bridge types. Other bridge types are evaluated as part of a bridge portfolio that is a result of multiple consultations with leading bridge designer companies. It contains bridges that are typical, commonly used or represent an important element of the transportation system.

4. Numerical model development

Non-linear numerical models for each bridge type are developed for both linear and non-linear dynamic analysis [S9]. The developed algorithm queries information from the bridge database to build the model based on the structural type, global geometry and element types with specific material models. The main structural elements are modeled with beam elements, while nonlinear springs are used to model the flexible supports, the soil-structure interaction and the bearings (Fig. 8). The model in its present form can handle the following situations: 1) multiple superstructures; 2) multiple piers in the transverse direction; 3) individual or common bents and abutments in case of multiple superstructures; 4) individual or common foundation for the piers and abutments; 5) skewness of the superstructure as well as of each
support; 6) discontinuity of the superstructure at any point (e.g. expansion joint at the supports).

Table 3 Summary of bridge components, applied elements and material models.

<table>
<thead>
<tr>
<th>Component</th>
<th>Code</th>
<th>Element</th>
<th>Material model</th>
<th>Modeling description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Superstructure</td>
<td>SS</td>
<td>elasticBeamColumn</td>
<td>Elastic</td>
<td>Elastic properties</td>
</tr>
<tr>
<td>Pier cap/tie beam</td>
<td>PC</td>
<td>elasticBeamColumn</td>
<td>Elastic</td>
<td>Elastic properties</td>
</tr>
<tr>
<td>Monolithic joint 1</td>
<td>MJ1</td>
<td>ZeroLength</td>
<td>Elastic</td>
<td>Rigid in all directions</td>
</tr>
<tr>
<td>Monolithic joint 2</td>
<td>MJ2</td>
<td>ZeroLength</td>
<td>Pinching4</td>
<td>Cyclic response of rebar</td>
</tr>
<tr>
<td>Elastomeric bearing</td>
<td>EB</td>
<td>ZeroLength</td>
<td>Steel01</td>
<td>Cyclic response of friction</td>
</tr>
<tr>
<td>Conventional bearing</td>
<td>CB</td>
<td>ZeroLength</td>
<td>Steel01</td>
<td>Stiffness: elastomer; Yielding: dynamic friction</td>
</tr>
<tr>
<td>Expansion joints</td>
<td>EJ</td>
<td>ZeroLength</td>
<td>ElasticPPGap</td>
<td>Rigid stiffness with gap (only in compression)</td>
</tr>
<tr>
<td>Piers</td>
<td>P</td>
<td>dispBeamColumn</td>
<td>Concrete01</td>
<td>Unconfined concrete fibers</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Concrete01</td>
<td>Confined concrete fibers</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Steel01</td>
<td>Steel rebar fibers</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Steel01</td>
<td>Steel rebar fibers</td>
</tr>
<tr>
<td>Abutment</td>
<td>Ab</td>
<td>elasticBeamColumn</td>
<td>Elastic</td>
<td>Elastic properties</td>
</tr>
<tr>
<td>Backfill soil</td>
<td>BS</td>
<td>ZeroLength</td>
<td>ElasticPPGap</td>
<td>Passive earth pressure only in compression</td>
</tr>
<tr>
<td>Shallow foundation</td>
<td>FS</td>
<td>SoilStructure</td>
<td>Elastic</td>
<td>Translational and rotational linear springs</td>
</tr>
<tr>
<td>Pile foundation</td>
<td>FP1</td>
<td>ZeroLength</td>
<td>Elastic</td>
<td>Elastic pile model (dispBeamColumn alternative)</td>
</tr>
<tr>
<td>Pile foundation</td>
<td>FP2</td>
<td>elasticBeamColumn</td>
<td>TzSimple1</td>
<td>Skin friction model</td>
</tr>
<tr>
<td>Pile foundation</td>
<td></td>
<td></td>
<td>QzSimple1</td>
<td>End compression model</td>
</tr>
<tr>
<td>Pile foundation</td>
<td></td>
<td></td>
<td>PySimple1</td>
<td>Lateral behavior model</td>
</tr>
</tbody>
</table>

![Fig. 8 Schematic illustration of the general beam element model.](image)

![Fig. 9 Typical bridge configurations: a) PMG-I; b) PMG-NI; c) SLAB; d) other conventional girders.](image)

The component modeling technique is dependent on the examined bridge type. The applied element types and material models are summarized for each component in Table 3, and Fig. 9 illustrates modeling approaches for typical bridge classes.

Piers are expected to suffer plastic deformations during a seismic event. Besides the cyclic non-linear behavior of the piers, 4 important component modeling techniques (which are not common in practice, but can alter significantly the seismic responses) are also investigated. Pounding between the bridge components (e.g. at the expansion joints) is incorporated in the
A numerical model is worked out for monolithic joints with shear reinforcements (Fig. 10b): a bilinear concrete-concrete frictional model is connected parallel to a pinching material model calibrated to laboratory test data found in literature (Fig. 10c) [S3].

The support of the backfill soil may be significant, especially in case of integral abutments. This behavior is modeled with non-linear springs attached to the nodes of a rigid grid modeling the surface of the abutment (Fig. 11a-b). Additionally, a Beam on Nonlinear Winkler Foundation approach is worked out for pile foundations (Fig. 11c) where parameter definitions are proposed in line with Eurocode 7 and Hungarian practice [S12].

Typical configurations of the examined PMG-I and SLAB bridge types are shown in Fig. 12 and 13. The input parameters are determined with the statistical analysis of the existing bridge database. These bridges are highly popular structures; some structural attributes (pier...
cross-section, abutment layout, pier cap cross-section) are fixed, while other parameters (e.g. height of the superstructure, transverse distance of the girders) vary with the span length (Table 4).

![Image](image1)

**Fig. 12** General layout of PMG-I bridges: a) side-view of the bridge; b) typical cross section; c) applied pile foundation arrangements for different bridge widths.

![Image](image2)

**Fig. 13** General layout of SLAB bridges: a) side-view of the bridge; b) general cross-section.

**Table 4** Input parameters for the parametric study (parameters for fragility analysis are marked with red fonts).

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Width</th>
<th>Notations</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. of spans</td>
<td>8 m</td>
<td>14 m</td>
</tr>
<tr>
<td>Pier height</td>
<td>2-4</td>
<td>7-8-10 m</td>
</tr>
<tr>
<td>Span length (L)</td>
<td>5-10</td>
<td>15-25-30 m</td>
</tr>
<tr>
<td>SS height</td>
<td>$h_{ss} = -0.0004 (L-5)^2 + 0.035 (L-5) + 0.4 [m]$</td>
<td></td>
</tr>
<tr>
<td>No. of piers in the transverse direction</td>
<td>3</td>
<td>4</td>
</tr>
<tr>
<td>Foundation - abutment</td>
<td>1x4 D=80 cm</td>
<td>1x6 D=80 cm</td>
</tr>
<tr>
<td>Foundation - pier</td>
<td>2x4 D=80 cm</td>
<td>2x6 D=80 cm</td>
</tr>
</tbody>
</table>

Assuming typical material properties and reinforcements, the standard resistance and the maximum acceptable PGA (causing total utilization of the component) is determined for each component of each configuration. For each bridge, a design PGA value is assigned based on their known locations, and component demand to capacity (DC) ratios are calculated by comparing the design PGA value to the maximum acceptable PGA value.

To see which components are critical, and how many bridges have specific critical components, empirical cumulative distribution functions for the component DC ratios are created and presented in Fig. 14.
The results show that in case of both single and multi-span bridges, the superstructure, backfill soil, abutment and foundations are adequate. For single span bridges, the superstructure-abutment monolithic joint is critical. The resistance of this component is insufficient for the majority of multi-span bridges as well, where pier shear and flexural failure is possible for 20-30% of the structures. The critical bridges are marked in Fig. 15. These maps are useful tools to identify critical bridges and to select regions of interest for a possible retrofitting project.

6. Fragility analysis

6.1. Adopted method for fragility analysis

Multiple Stripes Analysis (MSA) is adopted to create analytical fragility curves (Fig. 16a). At each intensity level, a number of ground motions are selected for each horizontal direction. Maximum demands are registered during non-linear time-history analysis, then the distribution of the demands (internal forces, deformations) is compared to the distribution of the capacity to calculate the probability of exceedance. These probabilities are plotted against the intensity measure to obtain the fragility curve [14]. This curve represents conditional probability; using the hazard curve of the site (Fig. 16b) the unconditional probability of failure can be calculated from which the reliability index ($\beta$) can be derived. In this study, the reliability index is used to compare the seismic performance of different bridge configurations.

Three damage limit states are considered that can be associated with the Damage Limitation (LS1), Significant Damage (LS2) and Near Collapse (LS3) LSs of EC8-3. The capacities of the bridge components are assumed to follow a LN distribution per EC0.

The fragility evaluation is divided into two main parts [S3],[S15]: 1) parametric study (see Table 4) for PMG-I bridges regarding their contribution to the bridge stock; 2) individual analysis of 30 representative bridges in the portfolio.
6.2. Effect of modeling and analysis assumptions

Before the fragility analyses, the effect of various modeling and analysis assumptions are investigated. Fig. 17 shows the importance of modeling the cyclic behavior of the monolithic joints. The developed cyclic model can take into account the degradation of the shear reinforcement. Failure at the abutment causes the redistribution of the seismic forces; pier demands are increased.

In Fig. 18, effect of the pounding at the expansion joint is illustrated: displacements of the superstructure are limited leading to decreased demands of the piers. In the meantime, the collision causes a pulse-like peak in the backfill soil deformations.
According to Fig. 19a, the derived fragility curves are highly dependent on the spectral shape applied for the analysis. It can also be observed that there is only a slight difference between fragility curves created for different sites (Budapest and Komárom).

The seismic responses may be sensitive to even a small variation of specific input parameters. The geometry, materials and soil stiffness values are uncertain. Fig. 19b shows the effect of handling these input parameters as random variables and confirms that the geometric uncertainty has negligible effect, while the material and soil uncertainties may significantly influence the shape of the fragility curve and the calculated probability of failure.

In literature, a power law function (leading to a linear curve in the log-log space) is often used to describe the demands as a function of the intensity measure (Fig. 20c-d). The advantage of the applied MSA method is that median and standard deviation can be calculated at each IM level providing a better estimation for the responses against the IM (Fig 20a-b). In some cases (e.g. pier shear forces in Fig. 20b) the linear approximation is correct; however it is not appropriate to describe elements with high plastic deformations at higher intensity levels (e.g. pier reinforcing steel deformations in Fig. 20a).
Theoretically, the calculated probability of failure is independent of the chosen IM if hazard consistent ground motions are selected such as in this research. However, the appropriateness of other IMs is evaluated to provide guidance in case of an arbitrary record selection. The fragility curve should be derived as a function of that IM which is in a high correlation with the examined seismic responses. This correlation is evaluated for various intensity measures for all bridges in the study. For instance, Fig. 20e shows that the optimal IM is highly correlated with the dominant fundamental period of the structure controlling the selected response. Accordingly, it is concluded that PGA, acceleration spectrum intensity (ASI) and Arias intensity (AI) describe well the demands of both PMG-I and SLAB bridges due to their low fundamental periods.

6.3. Evaluation with fragility curves

The component fragility curves are useful to highlight critical components, and to calculate the probability of component failure, while a system fragility curve is used to determine the probability related to the whole structure. The parametric fragility analysis confirmed the conclusions of the preliminary studies for PMG-I and SLAB bridges. LS1 is reached with the yielding of the abutment joint; LS2 is either pier flexural or shear failure, while collapse is caused by shear failure of the pier (flexural behavior is usually better) (Fig. 21).

![Fig. 21 Fragility curves for a typical PMG-I configuration. a) LS1; b) LS2; c) LS3.](image-url)

The insufficient pier shear resistance is common for other bridge types as well (precast multi-girder bridges with elastomeric bearings, reinforced concrete and composite bridges with conventional bearings). The high vulnerability of PMG-NI bridges is illustrated in Fig. 22. Fragility curves show that pier shear failure develops prior to any other component damage.

![Fig. 22 Fragility curves for a PMG-NI example bridge: a) LS1; b) LS2; c) LS3.](image-url)

Typically, steel bridges are exceptions, Fig. 22 illustrates that steel girders typically have an optimal behavior: pier flexural damage characterize all three damage limit states, energy dissipation due to the cyclic behavior of the piers can be utilized until collapse.
The bridge shown in Fig. 24 should be highlighted having special construction technology and layout: monolithic joints are used at the abutments, while the piers are restrained in the longitudinal direction, and free movements can develop in the transverse direction. Due to the supporting role of the abutment-backfill soil system, seismic pier demands are minimal in the longitudinal direction, besides, negligible horizontal forces are transferred to the pier by bearing friction in the transverse direction. Accordingly, the critical components are the abutment joints in LS1 and LS2, and it is also possible that failure is caused by unseating of the superstructure (see Ab. joint. trans. in Fig. 25).

Fig. 24 Special bridge configuration with special bearing arrangement.

6.4. Reliability levels of the structures

The $\beta$ reliability index for 50 year reference period is calculated for the area of Komárom (highest seismicity) and Debrecen (lowest seismicity) to provide a range of possible reliability indices for typical road bridges in Hungary [S3],[S7] (Fig. 26). Considering the minimum target reliability index value (1.98) proposed in literature [15], it can be concluded that precast multi-girder (PMG-I) and composite (COMP), reinforced concrete box girder (PC-B) and steel (STEEL) bridges with conventional bearings reach the minimum target value in most cases. Generally, most SLAB and precast multi-girder bridges with both monolithic joints and elastomeric bearings perform worse. In some cases, (see bridge 24 and 25 in Fig. 26), the calculated reliability index is unacceptably low which stems from the lack of seismic design and the improperly chosen structural configuration. For instance, the bridge 24 employs two circular piers in the transverse direction without tie beams and with low shear reinforcements.
The total length is 115 m, while only one support is restrained in the longitudinal direction. This means that one pier supports half of the mass of a 115 m long superstructure. Moreover, torsion develops from the transverse vibration causing additional longitudinal shear forces in the pier pairs at the longitudinal restrained supports. This configuration is a typical example where mere strengthening is not a reasonable option, the whole bridge behavior should be altered, conceptual seismic retrofit design should be performed. In Fig. 26, note also the high seismic performance of the earlier presented special bridge configuration of bridge 23.

Fig. 26 Range of reliability indices ($\beta$) related to collapse of the portfolio bridges.

To estimate what reliability level can be attained with seismic design, an intensity based evaluation using MMRSA is conducted to calculate DC ratios associated with pier shear failure (for all examined bridges), then these DC ratios are plotted against the corresponding reliability indices. According to Fig. 27, with seismic design ($DC \leq 1.0$) per EC8, a reliability index of ~2 can be reached.

Fig. 27 Reliability indices against the DC ratios calculated at the design PGA level.

7. Evaluation of possible retrofit methods

There are two basic approaches to retrofit a bridge [S4],[S11]: 1) conventional strengthening where the sole purpose is to increase the capacity; 2) demand mitigation with the alteration of the fundamental period, redistribution of seismic load and energy dissipation.

7.1. Conventional strengthening and seismic isolation

To evaluate the effectiveness of different retrofit methods, fragility analysis is conducted for various retrofitted configurations of two example bridges (BR03 and BR24) which represent typical configurations with typical problems. The main goal is to reach a reliability index of ~2 for the area of Komárom.

The first example is a PMG-I bridge (Fig. 28) where the monolithic joint at the abutment has to be strengthened by drilling holes in the concrete, positioning the reinforcement and finally injecting the holes (RF1). The cost is relatively high, since during the procedure the pavement has to be removed and then rebuilt. Collapse is caused by pier shear failure. Conventional strengthening methods can be applied efficiently, if the structural configuration
is chosen properly, but a moderate increase in the resistance is needed. In case of the example bridge, the original 1.82 reliability index can be increased to the minimum level with applying carbon fibre reinforced polymer (CFRP) strips to the piers (RF1, Fig. 29). However, after RF1, pier shear failure is still dominant, to obtain a more favorable ($\beta=2.83$) flexural failure mode, the shear resistance can be increased only with concrete overlay (RF2). The application of CFRP is cost-efficient and space-saving solution for typical PMG-I and SLAB bridges (where the increase of cross-section is limited); while concrete overlay can be used for bridges where the efficiency of CFRP may be drastically decreased due to the cross-section size and proportions (more robust piers).

Fig. 28 Original and retrofitted configurations of the example bridge with monolithic joints.

Fig. 29 Fragility curves for the examined configurations: a) PMG-I bridge; b) COMP-B bridge.

Fig. 30 Original and retrofitted configurations of the example bridge with conventional bearings.

For bridges where mere strengthening is not a cost-efficient solution (e.g. the retrofit of river bridge piers and foundations should be avoided) or the structural system is unfavorable, the seismic behavior should be altered to mitigate the demands. Such is the case of the second example bridge (Fig. 30). Various retrofit strategies are evaluated: RF1) distribution of the seismic forces among the piers; RF2) longitudinal seismic isolation; RF3) longitudinal and transverse seismic isolation. It is shown that the application of seismic isolators is an economical solution; the reliability index can be increased considerably (Fig. 29; from $\beta=0.65$ to $\beta=3.14$) while the costly pier and foundation strengthening can be avoided.

7.2. Seismic isolation design with equivalent linear analysis method

It is shown that the application of seismic isolators is a cost-effective retrofit solution if higher reliability can be achieved only with the alteration of the seismic behavior. The design
of such devices is a complex task: their non-linear behavior and the energy dissipation can be taken into account with linear response spectrum analysis (which is the most commonly used method in practice) only in an approximate way. For this reason, the accuracy of the equivalent linear analysis (ELA) method of Eurocode 8-2 is evaluated with non-linear time-history analyses. The equivalent linear characteristic is expressed with effective stiffness and effective damping to consider the non-linear behavior and the energy dissipation, however, an iteration procedure is needed to arrive at compatible forces and deformations (Fig. 31).

![Flowchart of the iteration procedure for equivalent linear analysis.](image)

Three typical bridge configurations are investigated and 48 ELA are conducted [S13]. Fig. 32 shows that the expected relative error is 20-30% and 50-75% for the internal forces and deformations, respectively. A methodology using a correction factor is proposed for conceptual seismic isolation design. The applicability of the method is verified through the retrofit design of an existing large span river bridge [S2].

### 7.3. Seismic design concepts for new bridges

Based on the preliminary and fragility analyses and the evaluation of retrofit strategies, design concepts are given for new bridges (due to the lack of space, details are presented only in the dissertation). With the proposed design concepts, economical structures can be designed and at least a reliability index of ~2 can be reached.
8. Summary

The seismic performance of Hungarian road bridges is not known, however their failure would cause significant economic consequences. A framework is worked out as part of the research to carry out the seismic performance evaluation of road bridges in a national level. Unfortunately, the existing bridge database does not provide sufficient information for reliable numerical modeling of each structure. Thus, bridges are grouped into classes and a bridge portfolio of typical bridges is created for further analysis. The critical components and configurations of the selected bridge types are determined; their fragility curves are created and their reliability indices are also calculated. Additionally, different retrofit strategies are evaluated for typically critical bridges; and seismic design concepts are proposed for new structures. The results contribute to three main topics: 1) seismic analysis and design; 2) seismic performance evaluation; 3) seismic retrofit design.

The study showed that the reliability level is low in case of several typical road bridges in Hungary. There is a need for a broader evaluation in order to make economic and financial decisions on a national level. Therefore, the extension of the database and the seismic performance evaluation of the whole bridge inventory are primary tasks in the future.

9. New scientific results

Thesis I  Related publications: [S1],[S6],[S16]
I investigated the seismic hazard of Hungary, and carried out comparative evaluation of site specific spectra and Eurocode 8-1 standard spectra.

I/a I employed a probabilistic seismic hazard analysis procedure and adopted a record selection method based on the general conditional intensity measure approach for Hungarian sites. I also created a freely available artificial record generation program.

I/b I showed that earthquakes with moderate magnitude (<5.5) and small epicentral distance (<10 km) contribute most to the seismic hazard of Hungarian sites at the design intensity level; and that the expected significant duration of ground motions is under 10 s.

I/c I showed that the Eurocode 8-1 Type 2 standard spectrum describes the site specific spectra better in Hungary than the Type 1 spectrum.

I/d I evaluated different intensity measures, and I verified that the seismic response of bridges is highly correlated with the spectral acceleration and the spectral intensity associated with the dominant vibration period controlling the seismic response.

Thesis II  Related publications: [S5],[S7]
I developed an evaluation framework to determine the seismic performance of Hungarian road bridges based on fragility analysis.

II/a I worked out five separate (database, seismic load generation, numerical model generation, seismic analysis and post-processing) modules in detail, and automatized the evaluation procedure by creating connection between these modules.

II/b I investigated the applicability of the existing bridge database for the seismic performance evaluation of the whole bridge inventory. I highlighted the shortcomings of the database; and proposed an extended database structure and a strategy for the extension.

II/c I classified the bridges into 8 bridge classes and characterized their most important structural attributes. I created a portfolio of 30 representative existing bridges.
**Thesis III  Related publications:  [S14],[S15]**

I conducted a preliminary parametric study for precast multi-girder and slab bridges with monolithic joints using linear modal response spectrum analysis.

III/a I developed the linear numerical model of the structures; and proposed a modeling technique for the backfill soil in case of linear modal response spectrum analysis.

III/b I determined critical configurations and components of typical bridge structures, and showed that the performance of the superstructure, abutment, backfill soil and the foundation is adequate; and that the monolithic joints are critical, especially at the abutments. I also showed that in case of short (<5 m) and high piers (>5 m) of longer (>50 m) bridges, the pier shear and flexural resistance are insufficient, respectively.

III/c I estimated the number of critical integral precast multi-girder and slab bridges in the inventory based on the standard evaluation per Eurocode 8-2.

**Thesis IV  Related publications:  [S7],[S9],[S12],[S15]**

I carried out a parametric fragility analysis for precast multi-girder bridges with monolithic joints; and conducted fragility analysis of 30 structures representing typical bridge types of the Hungarian bridge stock.

IV/a I developed the nonlinear numerical model of each bridge class. I worked out and calibrated a numerical model for the cyclic nonlinear behavior of monolithic joints with shear reinforcement. I developed a Beam on Nonlinear Winkler Foundation model for pile foundations; and proposed parameter definitions in line with Eurocode 7 and Hungarian practice.

IV/b I evaluated the effect of different modeling and analysis assumptions on the seismic response of bridge structures. I showed the importance of modeling the cyclic behavior of monolithic joints and the pounding between bridge components; and showed that the geometric uncertainty is negligible, while the material and soil uncertainties may significantly influence the calculated probability of failure.

IV/c I determined the most vulnerable bridge components using the component fragility curves. I verified the conclusions of the preliminary study for precast multi-girder and slab bridges, and also showed that pier shear failure is dominant for several bridge configurations. I pointed out that steel bridges with conventional bearings have better behavior, the collapse may be characterized with pier flexural failure.

IV/d I evaluated the reliability of typical road bridges, and compared the seismic performance of the structures based on their reliability index. I showed that an improper structural system and the lack of seismic design may result in an extremely low reliability index; and showed that seismic design per EC8-2 leads to a reliability index of around 2.

**Thesis V  Related publications:  [S2],[S11],[S13]**

I proposed retrofit methods for existing vulnerable structures and design concepts for new bridges.

V/a I showed that applying monolithic joints at the abutments, and conventional fix and free bearings in the longitudinal and transverse directions at the piers is an optimal configuration for highway overpass bridges up to 100 m total length.

V/b I evaluated retrofit strategies for two representative critical bridges. I showed that carbon fiber reinforced polymer is an optimal space-saving solution for pier strengthening when only moderate increase of the reliability is required; while
significant increase can be achieved with concrete overlay. In case of an improper structural system, seismic isolation is a cost-efficient retrofit method.

V/c I evaluated the accuracy of the equivalent linear analysis method of Eurocode 8-2 with non-linear time-history analyses; and proposed a methodology for conceptual seismic isolation design, which I verified through the retrofit design of an existing large span river bridge.

10. Publications of the author on the subject of the thesis

International journal papers

Hungarian journal papers

Papers in edited book

Conference papers
11. References