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Article

Post-Earthquake Rapid Damage Assessment of Road Bridges in Glina County

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Abstract: In December 2020, a strong earthquake occurred in Northwestern Croatia with a magnitude of M_L = 6.3. The epicenter of this earthquake was located in the town of Petrinja, about 50 km from Zagreb, and caused severe structural damage throughout Sisak-Moslavina county. One of the biggest problems after this earthquake was the structural condition of the bridges, especially since most of them had to be used immediately for demolition, rescue, and the transport of mobile housing units in the affected areas. Teams of civil engineers were quickly formed to assess the damage and structural viability of these bridges and take necessary actions to make them operational again. This paper presents the results of the rapid post-earthquake assessment for a total of eight bridges, all located in or around the city of Glina. For the assessment, a visual inspection was performed according to a previously established methodology. Although most of the inspected bridges were found to be deteriorated due to old age and lack of maintenance, very few of them showed serious damage from the earthquake, with only one bridge requiring immediate strengthening measures and use restrictions. These measurements are also presented in this paper.

Keywords: earthquake; bridge; damage assessment; strengthening; rehabilitation

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1. Introduction

Last year, Northwestern Croatia was shaken by two major earthquakes. The first occurred in March 2020, with an epicenter 10 km north of the capital Zagreb and a magnitude of M_L = 5.5. The second occurred in December 2020 with an epicenter near the town of Petrinja 50 km southeast of Zagreb and a magnitude of M_L = 6.2. Both earthquakes caused devastating damage to buildings and other structures. The World Bank estimated the total damage to be around 16.5 billion euros for 73,000 affected buildings [1,2]. In the first earthquake in Zagreb, most of the damage was due to the old age and poor maintenance of the buildings over the last 100 years of their existence. These buildings date back to the beginning of the 20th century and were mainly constructed with masonry and timber structural elements [3]. Furthermore, they were built without any seismic design requirements, had undergone many unauthorized reconstructions and adaptations during their lifetime, and had not been properly maintained [4]. The second earthquake in Petrinja was much stronger (M_L = 6.2, VIII-IX EMS-98 [5]), with a specific fault mechanism and shallow focal depth. Surface failures that occurred showed damage to linear infrastructure along a 30 km long section of the NE–SW strike [5]. New fault planes occurred along the NW–SE Dinaric strike, activating the 20 km long section of the Pokupsko fault [5,6]. A complex fault system was activated at the intersection of the two main longitudinal and transverse faults (Petrinja and Pokupsko faults) [5]. The PGA (peak ground acceleration) values for the bedrock foundation ranged between 0.29 and 0.44 g, but due to the high nonlinearity of the soil that was composed of clays with medium-to-high plasticity (evident from surface deposits and significant ground fractures [7]), it was estimated that locally amplified PGA values were likely in the range of 0.4–0.6 g [5]. This was also consistent with the observed damage to the buildings, of which approximately 15% sustained severe damage or complete collapse (DG4 and DG5) (Figure 1), 20% sustained significant damage (DG3), and 65% sustained light-to-moderate damage (DG1 and DG2) [5]. These damage grades were assigned according to EMS98 [8,9]. The main earthquake (M_L = 6.2) was preceded by two foreshocks (M_L = 4.7 and 5.2) and a series of aftershocks (up to M_L = 4.9). A strong foreshock helped save lives, as many critical buildings had been evacuated before the main earthquake struck and caused them to collapse. Many significant aftershocks that occurred over the next month (though noticeable earthquakes of up to $M_L \approx 3$ are still recorded weekly even now, a year after the main earthquake) caused subsequent damage to already damaged structures, making it difficult to classify the extent of damage and demanding the reassessment of already examined structures. In contrast to the Zagreb earthquake (March 2020), the Petrinja earthquake (December 2020) showed significant damage to linear infrastructure and underlying soil (such as landslides, liquefaction, suffusion, and sinkholes) that occurred during or as a result of the earthquake over the next few months. Significant damage occurred to structures crossing the activated faults of the fault system, evident on roads, bridges, pipelines, and river embankments. Examinations of the road damage revealed dextral co-seismic strike-slip cracks and displacements [6]. The clearest evidence of an active Petrinja fault was the presence of cracks on the Brest Bridge on the Kupa River, which was under tension along the fault line [6]. Galdovo Bridge over the Sava River in Sisak showed a 10 cm abutment bearing displacement as a consequence of a still-unknown N–S fault line [6]. There was also damage to the pipelines and river embankments of the Kupa and Sava rivers. More than 90 sinkholes occurred within a radius of about 10 km without prior warning of ground deformation. Many of them had a radius of 25 m and a depth of 12 m and endangered the surrounding buildings and infrastructure [7]. Some of them are still active and make any reconstruction work impossible. After the Petrinja earthquake, structural engineers from all over the country were called upon by the Civil Protection Agency to assess the extent of the structural damage, safety, and restrictions on use, as well as to work in collaboration with emergency rescue and demolition services to mitigate the consequences of the disaster. Assessments were prioritized based on the extent of damage and the importance of the structures. Thus, health and infrastructure structures were assessed first. The assessment of bridges was particularly important due to the need for their immediate use by rescue teams working with heavy machinery that needed to be quickly and safely moved. Moreover, due to cold and snowy winter weather, a humanitarian crisis occurred as people evacuated their destroyed or damaged homes and had to be temporarily housed.

Figure 1. DG4 and DG5 damage from the December 2020 earthquake in Sisak-Moslavina county.

Mobile housing units had to be transported on a large scale to the affected areas, which required the use of special heavy vehicles and the crossing of many bridges. The entire area is located at the intersection of four rivers (Sava, Kupa, Glina, and Maja) and many of their tributaries, so many bridges had to be immediately checked for safety. Most of these bridges are more than 50 years old and therefore not designed for the seismic safety required by modern standards. In addition, many of them were already in poor state due to material deterioration, they had not been properly maintained, and they had been subjected to overweight heavy traffic for which they were not designed. There were also very little to no data of their prior examination and assessment, nor any documentation from their design. Many specialized teams of civil engineers with experience in bridge engineering were assigned to this task and sent to different regions of the affected area. This paper provides an overview of the post-earthquake assessment of bridges in the Glina region, which included eight overall bridges.

In the forensic investigation of bridge failures, a sequence of events was identified using an interdisciplinary information gathering approach to identify the main causes of failure [10]. This approach can also be used to determine the current condition of the structure following an event that was not anticipated when the bridge was designed [10]. The first and most important part of any forensic investigation is the visual inspection of a bridge, followed by the collection of existing information about the structure, followed by non-destructive testing (NDT), and finally the analysis of all data using numerical and/or analytical models to determine cause–effect relationships.

It is important to emphasize that the task of this assessment was not to provide a detailed account of the existing load-carrying capacity of the bridge or its past deficiencies, as this could not be done without calculations and NDT for which there was no time in a crisis situation. The task was to identify critical damage as a result of the earthquake, assess the possibility of continued use of the bridge, and establish restrictions and guidelines for the use of the bridge. This work relied heavily on the experience of the commissioned engineers and their good judgment.

2. Theoretical and Practical Background in Bridge Assessment

2.1. Bridges Visual Inspection—Practices Overview

Visual inspection is a fundamental tool for bridge assessment and decision making (Figure 2). The visual inspection of a bridge greatly differs from that of any other structure due to bridges' generally longer life span, exposure to very aggressive environmental conditions, and structural elements made of different materials with different deterioration processes and rates. Improper and untimely maintenance leads to rapid changes in the slope of the time-related deterioration curve that determines the remaining service life of individual bridge elements [11,12], and the failure of any non-structural element (such as waterproofing, drainage, and expansion joints) is critical to the duration of the remaining service life.

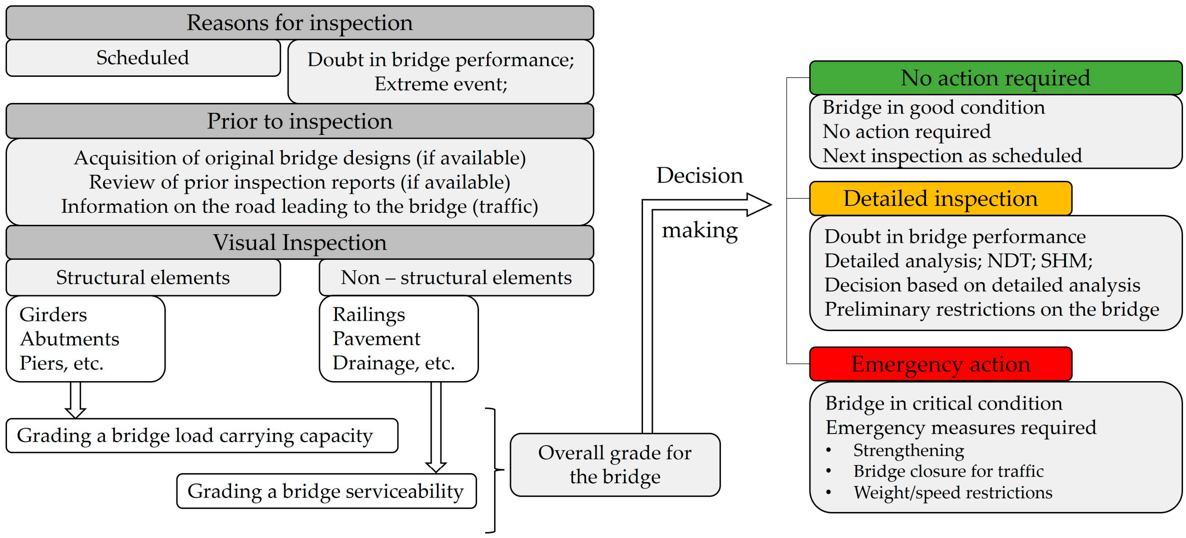

Figure 2. Flowchart for visual inspection protocol and decision making.

Most bridge visual inspections are based on a rating system in which a bridge is divided into elements and each element is assigned a numerical condition state describing its degree of damage. Condition states ranging from "no defects" to "critical defects" often comprise 5 or 6 rating points [13–16], but there are also examples of up to 10 rating points [16]. In a more detailed visual inspection, rating points are assigned not only to bridge elements but also to a location within an element, thus creating a geometrical mesh of damage distribution [13]. Since the damage location determines the failure mechanism, such an approach is beneficial for structural reliability analysis. It is imperative that the point ranking system for each visual inspection procedure provides a detailed description for each point of the condition assessment and that this description of damage is done separately for each bridge element and material. The collected data can be sorted and analyzed using mathematical statistical methods. Then, using probability-based models, one prioritizes the extent and timeframe of maintenance work [14]. A 2002 study [17] examining the reliability of visual inspection of highway bridges concluded that there was a significant spread in ranking points between inspectors, with only 68% of inspectors differing by up to one rating point within a 10-point rating system. This spread was attributed to the inspectors' individual formal training, the thoroughness of the inspection, and (in part) their subjective perception of the significance of the damage. Other limitations and shortcomings of any visual inspection can be summarized in three categories [18]: 1. timing (recognizing the damage at the moment it occurs and detecting the damage propagation rate in time); 2. interpretability (subjective evaluation by different inspectors depending on their training and given guidelines); and 3. accessibility (ability to access all elements and the interior of the structure to detect damage). To mitigate some of these shortcomings, a combination with non-destructive testing (NDT) [19] and structural health monitoring (SHM) [18] is recommended.

In order to obtain useful information for planning appropriate maintenance work, a visual inspection must be standardized within a certain management system and certain documented guidelines for an inspection procedure and frequency must be provided [18]. A 2010 study conducted by the Croatian Road Administration to assess bridge condition based on visual inspections introduced a six-category rule ranging from 0 (undamaged) to 5 (extensive damage) [14]. For each bridge, twelve bridge elements were evaluated, divided into three element groups (substructure, superstructure, and equipment). An example of visual inspection as a tool for evaluating bridge performance and prioritizing bridge repair in the transportation network can be found in [20]. The defined method was applied to six different (in terms of length and structural system) bridges in Croatia.

Although visual inspection is the imperative in any bridge assessment methodology, a detail account of bridge performance can only be obtained by collecting additional data of the bridge structure. These data must include stiffness distribution parameters, material properties, real traffic loads, and modeling analysis [20,21]. For example, a very effective procedure of collecting additional performance indicators is bridge weigh-in-motion (B-WIM), a method that uses real traffic data to determine the effects of maximum load on a particular bridge and later decision making based on value of information (VoI) analysis [16,22]. Additional information can also be obtained through non-destructive testing to assess the damage in the reinforcement of RC bridges and subsequently predict their service life using numerical models [19].

The visual inspection conducted during this rapid assessment of the bridges in Glina county after the earthquake followed the methodology shown in Figure 2 with some modifications. These modifications were made due to the lack of information about the bridges that is typically collected prior to the visual inspection and the need to act very quickly and make decisions. The focus of the inspection was placed on the structural elements (including bearings) that are critical to evaluating the load-carrying capacity of a bridge. The serviceability rating of the bridge was not important in the decision-making process. Nevertheless, the deterioration of non-structural elements was recorded for future reference and is also presented. Since this quick visual inspection immediately after the earthquake only served to answer the question of whether the bridge should continue to be used after the earthquake, no scoring system was used and the ratings were given as "continue to use", "close the bridge", or "issue use restrictions" (Figure 3).

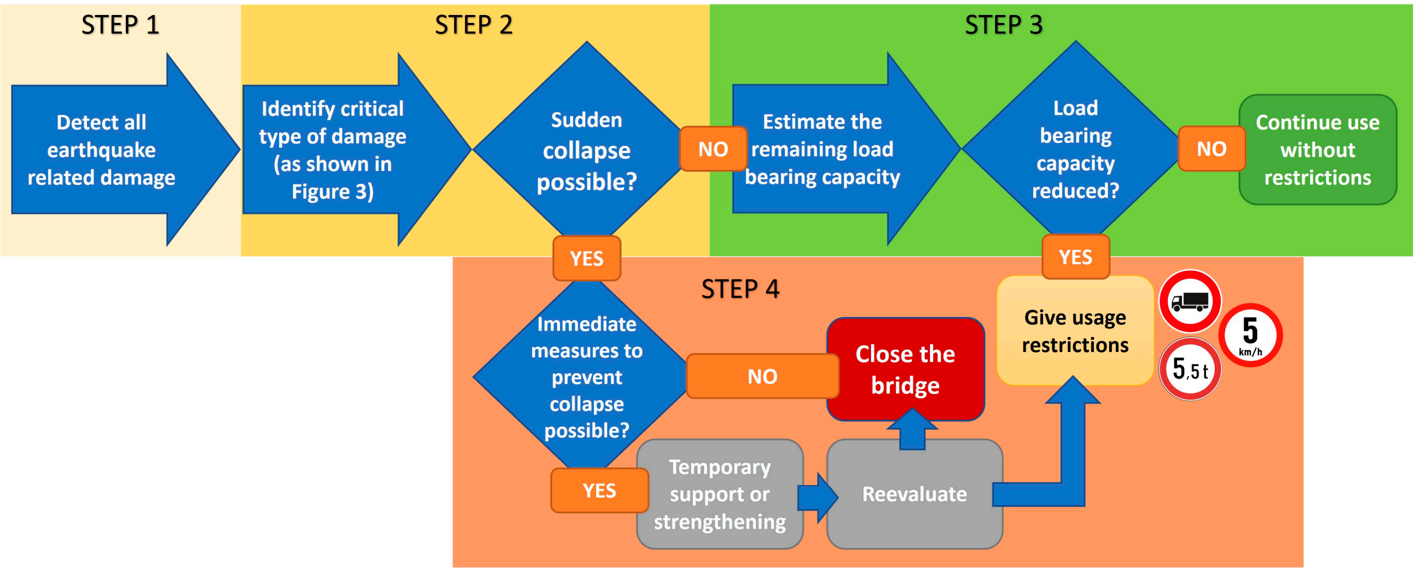

Figure 3. Post-earthquake bridge rapid assessment methodology.

2.2. Bridge Seismic Assessment Methods

There are a number of methods for the seismic assessment of existing road bridges, depending on the degree of complexity and practicality. There is no universal opinion on which is the optimal method, as this depends on a number of parameters mainly related to the characteristics of the bridge (structure, span, material used, etc.).

In general, it can be said that nonlinear analysis is more suitable for existing bridges because it considers the plastic behavior of the elements. Most often, a performance-based assessment such as nonlinear pushover static analysis or nonlinear time history dynamic analysis is used, which utilizes the ductility and energy dissipation characteristics of the real structural behavior. The pushover method measures structural capacity through inelastic displacement, which is then compared to the demands of a particular earthquake ground motion from the response spectrum. The accuracy of such an assessment largely depends on the accurate characterization of the material and dynamic properties of the bridge, which can be experimentally determined using destructive and non-destructive testing [23]. As pointed out in [23,24], this is particularly advantageous for traditional masonry bridges, whose material and dynamic properties are difficult to predict. For reinforced concrete arch bridges, a two-level seismic assessment procedure is possible [25], with evaluation checks at each level. The first, a more conservative level of evaluation uses a linear multimodal analysis, while the second level utilizes nonlinear pushover analysis for a less conservative, easier to meet safety requirement. The failure probability was investigated in a parametric study that included several variables such as geometry, material properties, earthquake records, and intensity levels [26,27]. The probability of failure was expressed by a safety indicator, which shows the difference between the seismic capacity and demand, obtained from a nonlinear dynamic analysis.

Based on the literature review and experience in the practical design and the assessment of existing reinforced concrete road bridges, a list of the most common deficiencies is further presented in this paper. The most critical element of bridges in seismic assessment is often found to be columns. The main causes of column structural deficiencies are a low percentage of longitudinal and transverse reinforcement, poor concrete, inadequate seismic detailing, and a lack of confinement reinforcement [28]. The ductility of columns is important for seismic energy dissipation, but for older bridges that do not comply with modern seismic design standards, it is very difficult to estimate the level of ductility. Research [29] has shown that the use of smooth rebar reinforcement, which is common in bridges older than 30 years, helps to improve the ductility of atypical cross-sections without modern seismic design. The failure mode of columns in earthquake situations is often found to be shear critical brittle failure due to the very limited shear capacity of short piers or flexural failure of tall columns [28]. A comparison of the prediction of shear strength capacity according to various codes models and experimental results for hollow circular piers was shown in [28]. A significant influence on the seismic response of a bridge is the relationship between the soil and the foundation. A rigid foundation-soil model may overestimate the seismic capacity of a bridge. This is especially important for masonry arch bridges. Research [30] has shown that that a 50% increase in safety confidence level can be observed when ignoring soil foundation flexibility effects.

For the earthquake assessment of multiple bridges on a larger scale, research has been carried out to develop the necessary tools to quickly determine the vibration periods for the structural seismic demand and fragility of structures, when only basic geometric variables are known [31]. To this end, a large database of reinforced concrete bridges was created and statistically processed to identify relationships between seismic response parameters and geometric and material input variables. Such relationships can assist in rapid assessment actions.

A holistic probabilistic framework [32] based on visual inspections and fragility curves has been proposed for the assessment of bridges in the network after an earthquake. Fragility curves have shown the relationships between the parameters of a seismic event (PGA, spectral acceleration, and measure of shaking intensity) and the probability of structural damage when the given performance level of a bridge is exceeded. This methodology uses a six step process to determine the interventions needed after a catastrophic event: 1. gathering information (visual inspections); 2. deriving fragility curves; 3. deciding whether a non-destructive evaluation is needed after an earthquake; 4. updating fragility curves for the damaged bridge while considering the uncertainties of visual inspection; 5. deciding whether to allow traffic to cross over a damaged bridge; and 5. deciding for immediate repairs [32]. Fragility curves can be used for the basic evaluation of multiple bridges on a section of a transportation network, but detailed analysis should be based on a more site-specific approach.

The decision-making process based on assessment results is often implemented in bridge management systems for priority rankings. In the case of seismic evaluation, the decisions are primarily based on comparisons between the fragility curves of a bridge with and without seismic retrofit measures. In [32], partial restriction of traffic on a damaged bridge after an earthquake was never considered due to the uncertainties related to the loss of load-carrying capacity. The decision to close a bridge is based on the ratio between the new updated risk of failure of a damaged bridge and the risk of failure before the earthquake event [31]. If this ratio is greater than 1, the bridge must be closed.

3. Petrinja Post-Earthquake Rapid Assessment Actions

3.1. Damage Assessment Management

After the earthquake in Petrinja, groups of volunteer civil engineers for crisis management were quickly formed and started operating throughout Sisak-Moslavina county. The formation and coordination of these groups was led by experts from the Faculty of Civil Engineering in Zagreb, whose previous experience from the Zagreb earthquake was crucial for rapid response and effective management. The methodology previously prepared based on Italian experience [33–35] included six-level classification categories: N1 and N2 as unusable buildings due to external risks or internal damage, respectively; PN1 and PN2 as temporarily unusable buildings due to uncertainties about the extent of damage requiring additional investigations or due to emergency remediation measures, respectively; and U1 and U2 as usable buildings without restrictions or with precautionary advice issued, respectively. Inspection groups were assigned to geographical locations and neighborhoods, and the results of their inspections (category classifications) were recorded via a centrally managed digital database system that was accessible via mobile devices and thus reflected an up-to-date situation in the terrain. However, this usability classification methodology and database did not consider or allow for other non-residential structures (such as infrastructure structures, special engineering structures, or bridges) to be included. Most of the inspection teams were educated by simple guidelines and given examples to use the grading categories for buildings only, leaving the more complex tasks of evaluating infrastructure structures to fewer groups of experts in their respective fields.

As noted earlier, the bridges needed to be assessed quickly because their availability was critical to many emergency services throughout the region. Previous experience in visual inspection, damage detection, and classification [19,20] was an important prerequisite for the bridge assessment team, as was experience with the seismic behavior of bridges [29]. Therefore, the teams with this practical knowledge were called in and conducted their assessments with the help of the road and transportation authorities.

3.2. Bridge Post-Earthquake Rapid Assessment Methodology

The methodology for rapid post-earthquake bridge assessment (Figure 3) in the case of the Petrinja earthquake was established on an emergency basis since there was no time to prepare, distribute, and discuss documented and detailed guidelines. It was imperative to keep bridges in service as long and as much as possible, closing them only when critical damage was detected. For bridges where moderate damage was found, it was recommended that operating restrictions (such as vehicle weight and traffic speed) be placed on their continued use. Where it was possible to provide emergency strengthening to a bridge to keep it operational in any capacity or prevent its complete collapse, services and resources were placed at a priority disposal for this work to be quickly carried out without the need for any design documentation.

The first step in the assessment was to identify all earthquake-related damage. Since none of the eight bridges assessed by this team had information on previous conditions or damage, it was important to identify the damage caused by the earthquake itself and distinguish it from any earlier damage. For example, fresh cracks in asphalt or concrete can be recognized when there is no water sediment or discoloration in or around the crack, fresh bearing displacements can be recognized by uncorroded scratch marks on bearing plates or blocks, abutment movements or rotations can be detected by cracks in the embankment soil or its erosion, and column movements or rotations can be detected visually.

The second step of the assessment was to identify the most critical type of damage that could lead to the collapse of the entire structure without any warning. The most obvious type of such collapse during or after an earthquake would be the slippage of the superstructure from the bearings, shear failure of the column or main girder, loss of stability of the substructure (overturning failure or sliding failure), or massive landslide erosion of the ground soil near the abutments (Figure 4). These types of earthquake bridge failures have been recorded in earthquakes in Japan and Chile [36–39], and all correspond to the bridge types found in this particular post-earthquake seismic assessment. Therefore, they were recognized as most likely to occur in these circumstances.

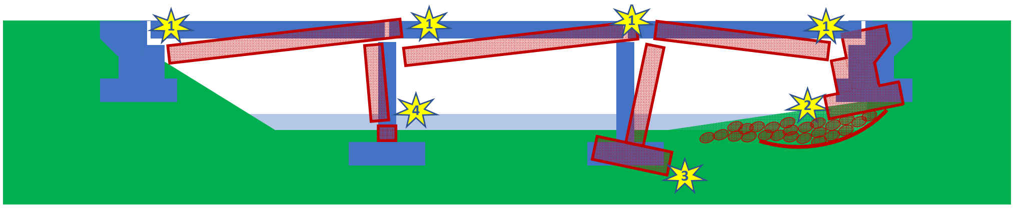

Figure 4. Most common earthquake types of bridge sudden collapses (any or all can occur): 1—bearing slippage; 2—abutment foundation soil landslide; 3—column turnover; 4—column shear failure.

The abovementioned types of collapse may occur independently, or they may be interconnected to form a progressive zipper-type collapse [10] in which the failure of the first element causes the failure of the second element, then that of the third, and so on, thus creating a cascading overall collapse of the bridge. Of course, the possibility of any or all of these collapse scenarios here depended on a bridge's structural system, location, and foundation type. These collapse scenarios could even occur in the coming days, weeks, or months after the main earthquake during the numerous small or moderate magnitude aftershocks that frequently occurred after the main Petrinja earthquake. They could even occur as a result of heavy traffic (axle loads or velocity-induced vibrations) on a structure or foundation soil that had reached a critically unstable equilibrium that is easily unbalanced over the tipping point. If such a possibility was detected, the bridge was to be immediately closed for traffic.

The third step was to evaluate the contribution of all cumulative damage to the reduction in the remaining load-carrying capacity of the bridge. This is undoubtedly the most difficult type of assessment as it had to be conducted without any testing or calculations. It could therefore only be given as an estimate, which had to be conservative enough for safety reasons but not too conservative as to unnecessarily hinder a much-needed use of the bridge in a crisis.

The fourth step of the evaluation was only required if the evaluation from the third step indicated a reduction in load-carrying capacity. In such a case, restrictions on bridge use, namely limits on vehicle axle loads, total vehicle weights, and maximum vehicle movement speeds, were required. In the event of a risk of further damage to the bridge or its sudden collapse, the final step of the assessment was to prescribe immediate measures to prevent this if possible at a given time with the available resources. It is obvious that this method of evaluation lacked the aspect of testing (destructive or non-destructive), static or dynamic analysis, and reliability calculations—all of which are required for any long-term seismic evaluation or seismic retrofit. This was, of course, due to the extreme circumstances of the crisis situation and the mitigation of consequences that would result from protracted decision-making or uncertainties in the required use of the bridge. Despite its shortcomings, this rapid assessment proved quick and effective, and it was undoubtedly a critical part of the post-earthquake emergency life-saving actions.

4. Glina Bridges Assessment and Damage Detection

After the Petrinja earthquake, different teams were deployed to examine bridges in the affected area within a radius of about 50 km. This paper presents the results of this examinations for bridges in and around the town of Glina, which covers a radius of about 10 km. This area is located 10–15 km from the epicenter of the earthquake and was therefore strongly affected by it. Figure 5 shows the map of this area and the total eight bridges that were examined (the numbering on the map follows the order of examination and is kept in the next subsections). All examinations were carried out in one day, with follow-up examinations for the most critical bridge (Section 4.2) in the next two days. Most of these bridges (Figure 6) were built in the previous century and are now 50 or more years old. There are only noted three exception bridges (numbered 1, 7, and 8 in Figure 4) that were built in the last 30 years. All bridges were found to have simply supported or continuous girders and have spans between 7 and 20 m. The superstructures were found to be either concrete slabs or composite steel-concrete ribbed section. The only exception was Glina Bridge (No. 1), which is a steel girder bridge with a span of 40 m. The main problem of all bridges, which became evident during the examinations, was the lack of maintenance, which, in combination with poor waterproofing, led to progressive material deterioration and subsequent damage to the structural and non-structural parts of the bridges. The most common types of this damage were concrete spalling, the corrosion of reinforcement, the corrosion of steel girders, the corrosion of railings, bearing degradation, the clogging of expansion joints, the cracking of asphalt, and the erosion of abutment slopes [40]. Most of these problems could have been avoided if timely maintenance had been performed to prevent further deterioration due to water intrusion and corrosion of the reinforcement. Although all these problems were evident and noted during the examination, the purpose of the examination was to record and evaluate any damage caused by the earthquake that would pose a risk to the continued use of these bridges and endanger the safety of the users. Therefore, it was necessary to accurately identify the nature of the damage according to its cause and significance to the overall load-bearing capacity and/or stability of each bridge. The visual inspection protocol and decision-making process, as shown in the flowchart in Figure 2, were followed as closely as possible. Obviously, a prior review of bridge documentation was lacking because it was not available for most bridges and/or in the critical timeframe. Grading system was only binary, i.e., the bridge was still to be used or it was to be closed and/or prescribed immediate action (as shown in Figure 4). Expert judgement was used for the safety assessment of the continued use of each bridge. A detailed inspection was one of the possible recommended measures, but it was not to be a prerequisite for further bridge use.

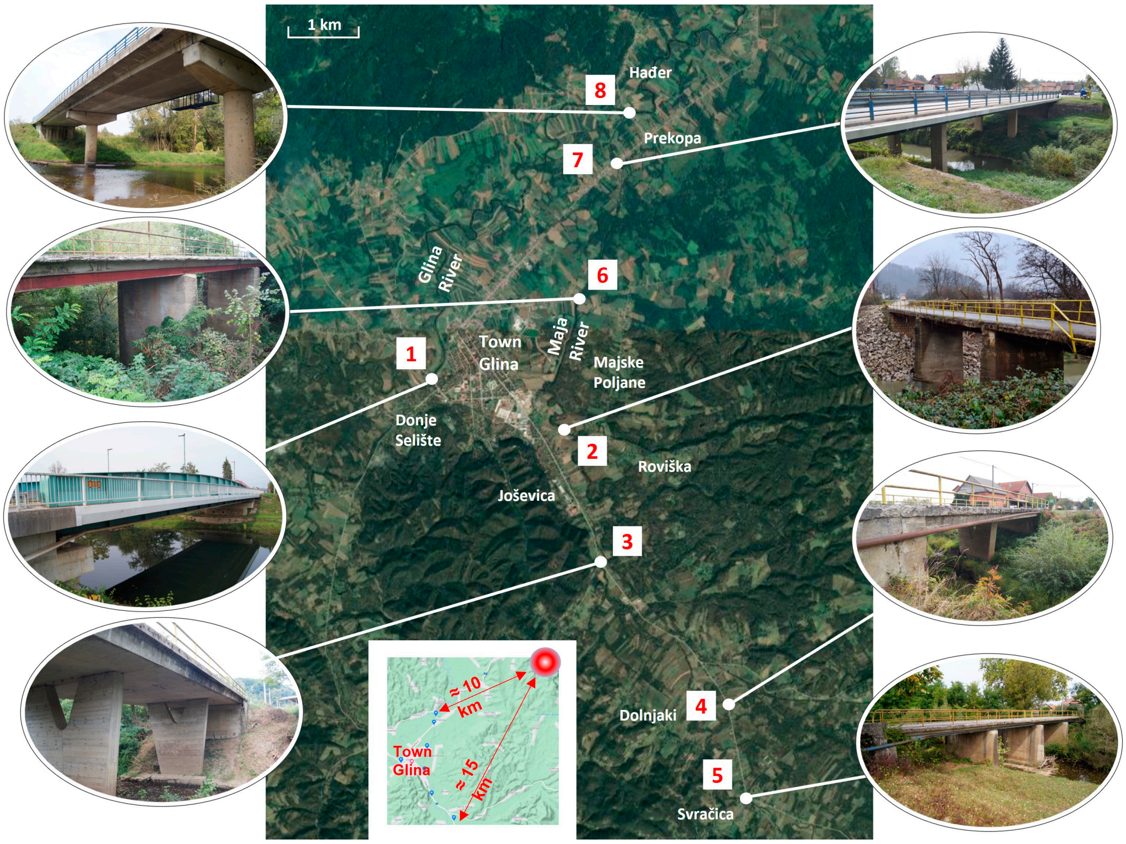

Figure 5. Geographical overview of Glina county bridges for post-earthquake rapid assessment.

4.1. Glina Bridge

Glina road bridge is the main city bridge over the river Glina, located in the south-west part of the town (Figure 5). It is the newest of all the examined bridges, built in 2003. It is a simply supported girder, crossing the river in one 42 m long span (Figure 6). The superstructure comprises two steel girders with variable height from 2.35 to 3.4 m and at 9.2 m apart (Figure 7a). The concrete road deck is supported by cross girders tapered in-between the main girders, and the footways are supported by consoles on the outside of the main girders. The deck slab is 20 cm thick on the carriageway and 12 cm thick on the footways. The overall width of the bridge is 12.6 m. Abutments are massive, 8.7 m high, reinforced concrete structures. Fixed bearing is positioned on the west abutment, and a movable bearing is on the east abutment. The bridge was found to be in an overall good condition, showing signs of medium structural steel and bearing corrosion, but no loss in the section area due to corrosion was detected. The concrete deck was found to be in almost perfect condition, with no reinforcement corrosion detected. Abutment concrete is also without any damage, there were only small parts of stone cladding detached. The partial erosion of the embankment slope around the abutment wings under the footways consoles was present, but it was not critical or caused by the earthquake.

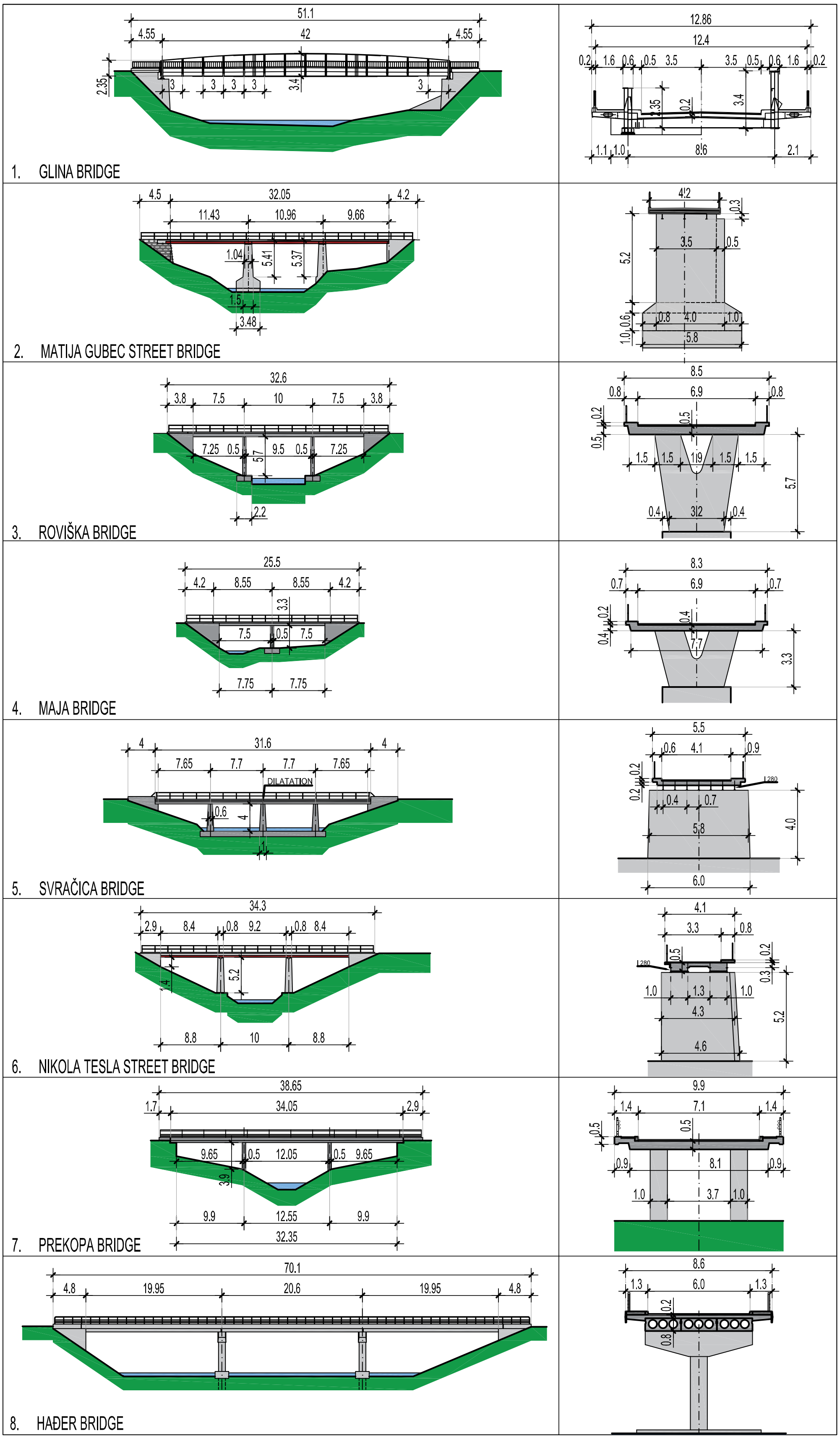

Figure 6. Glina bridges layouts and cross-sections (units in m).

Figure 7. Glina bridge: (**a**) main girder and carriageway; (**b**) fixed bearing; (**c**) movable bearing.

Examination showed no damage due to the earthquake. There were no signs of abutment movements; the fixed bearing successfully transferred the horizontal force on the abutment wall without any damage to the bearing or the wall (Figure 7b,c). Abutment embankments showed no landslide signs, and foundations showed no rotations or settlements. There was no visible damage to the expansion joints other than existing cracking in the asphalt layer due to dynamic loads from traffic. The expansion joints were found to be clogged with dirt and gravel and should be maintained in the future. No further actions were required, and the bridge was maintained for traffic use without any restrictions.

4.2. Matija Gubec Street Bridge

This bridge is located in the south part of the town, crossing the river Maja and leading to Majske Poljane village (Figure 5). The bridge superstructure is a series of simply supported girders over three spans: 11.43 + 10.96 + 9.66 m (Figure 6). The cross-section comprises three steel girders of 355 mm in height placed at 160 cm apart, as well as an 18 cm thick concrete deck (considering the age of the bridge, the level of section's composite behavior was unknown). The width of the superstructure is 4.15 m. The superstructure is directly supported by columns and abutments without any bearings. Abutments are massive structures, about 4 m high and 4.5 m long. It was evident that the bridge had undergone reconstruction in the past since one of the abutments was found to be a reinforced concrete structure and the other was found to be masonry structure from stone blocks. Its columns are massive, reinforced concrete structures that are 8 and 6.4 m high and 4 m wide. The east column was found to have visible scour signs, with parts of the foundation soil missing. The west column was shown to have a much wider and longer foundation (5.8 × 3.5 m) than the east column (4 × 2 m), which suggests that the west column foundation underwent a rehabilitation in the past. This was probably due to scour developing under the west column sooner due to its position in the middle of the riverbed (Figure 8a). It is evident that the bridge was in a poor structural state even before the earthquake: main girders were shown to be heavily corroded, the concrete deck slab was spalling, the reinforcement bars were visible and corroded, the edges of the footways and the cornice were eroded and largely missing, a permanent deflection in the superstructure was evident due to overweight traffic load, and the railings were not anchored in the footways. Maja river has a highly variable water level and flow speed that caused the erosion of the west riverbank and scour developing under the west and east column foundations (Figure 8a,d).

Figure 8. Matija Gubec street bridge: (**a**) columns—large foundation for the column in the middle of the riverbed and scour visible on the east column; (**b**) abutment stone wall damage; (**c**) abutment sliding signs; (**d**) evidence of subsequent abutment sliding—stone wall integrity compromised.

The examination revealed serious deficiencies in the west abutment. Stone wall joints showed cracks and openings up to few centimeters, with mortar missing and stone block movements (Figure 8b). The whole abutment showed signs of translation and rotation towards span opening due to ground movements and soil erosion (Figure 8c). The soil around the abutment wings showed signs of land sliding (Figure 8d). The best course of action at this time was to close the bridge, but this action would have severed the connection to the nearby settlement that was the most affected by the earthquake and needed supplies and help at this time. It was reluctantly decided that the bridge could stay open with restrictions of only 5 ton vehicles at 5 km/h traveling speed. Only one vehicle was permitted on the bridge at the same time. Furthermore, emergency actions were ordered to strengthen the abutment and prevent its further damage (see Section 5). The bridge was placed under continuous monitoring due to aftershocks that were frequent in the coming days. Subsequent inspection the following day showed further degradation of the abutment in which the falling of the stone blocks occurred and the abutment integrity was compromised (Figure 8d). At this time, it was decided that the bridge safety could no longer be assumed, and the bridge was completely closed for traffic. Stabilization measures were undertaken at this time.

4.3. Roviška Bridge

This bridge is located in the south access road to the town Glina (Figure 5). It is a reinforced concrete slab bridge with over three spans of 7.5 + 10 + 7.5 m (Figure 6). The superstructure comprises a 50 cm thick reinforced concrete slab that is directly supported by wide columns branching at the top. The width of the bridge is 8.5 m, columns are of variable cross-section between 3.2 × 0.5 m at the bottom and two branches (arms) at the top, each 1.5 × 0.5 m. The height of the columns is 5.7 m. The abutments are massive reinforced concrete structures with about 3–4 m high walls and 3.8 m long wings. Column foundations are 4.0 × 2.2 m slabs. There are no bearings present on the bridge and bridge is without any drainage elements. There was visible damage on the abutment walls, where the corner part of the side walls was found to be missing on both abutments, and reinforcement bars were protruding out of concrete (Figure 9a).

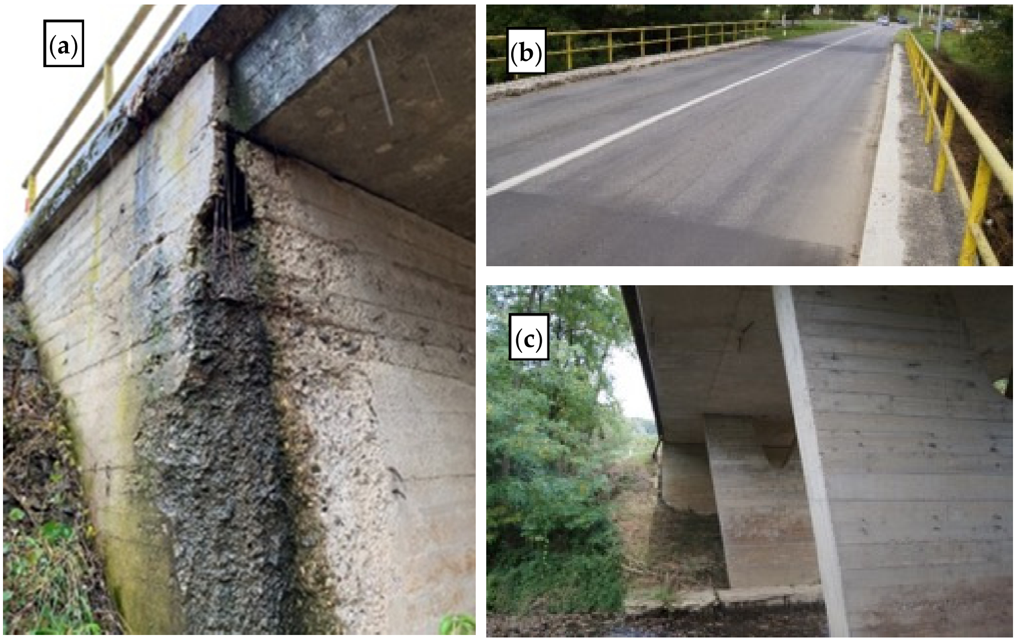

Figure 9. Roviška bridge: (**a**) damage to the abutment wall and wing, as well as drainage problems; (**b**) degraded footway (left) and new footway (right); (**c**) substructure without signs of earthquake damage.

Since the damaged parts of the wall showed heavy discolorations, traces of long-term water leakage, and algae sedimentations, it was evident that this damage was not caused by this earthquake. The quality of concrete in these fallen off parts of the abutment corners could be described as poor, exhibiting the local segregation of large fractions of aggregate and very low quantity of reinforcement. It is possible that this damage was a consequence of faults during erection since it resembled typical damage observed when improper concreating without vibration is performed. Since this damage was very localized and not in the main load transfer path, it was not considered serious at this stage of post-earthquake bridge evaluation. It did not compromise the load-bearing capacity of the abutment wall. One of the bridge footways and cornices were heavily degraded due to lack of waterproofing and reinforcement corrosion. The other footway had been rehabilitated in the past with a new concrete layer (Figure 9b).

Overall, the bridge was found to be in structurally good condition and showed no signs of damage due to the earthquake. There was no damage to the asphalt joint between the abutment and superstructure, and no displacements were recorded at the superstructure supports. Columns and abutments were without any major cracks, rotations, or settlements (Figure 9c). After the earthquake, the bridge was continued to be used for traffic without any restrictions. Further inspection and rehabilitation were recommended due to prior abutment wall damage, visible reinforcement corrosion, and possible scour on foundation piers.

4.4. Maja Bridge

Maja bridge is located just south of previous Roviška bridge (Figure 5), and it is of similar type but smaller length. The bridge is also a continuous girder slab bridge with over two spans of 7.5 + 7.5 m (Figure 6). The slab girder is 35 cm thick and 7.7 m wide, with short consoles on both sides. The superstructure is skewed in regard to abutments, columns, and the riverbed (Figure 10a). Height of the columns and abutments is about 3.5 m. There were no bearings or expansion joints causing dilatation cracks in the asphalt. There was also a visible vertical dilatation crack between the abutment wall and wing, suggesting that they were not fixed together and moved separately. The superstructure concrete was seen to be in relatively good condition, without signs of progressive reinforcement corrosion or concrete spalling but with visible signs of discoloration due to water leakage. Although drainage was present on the bridge, gutters were clogged and caused water to seep through the concrete slab. The slab consoles (footways) were observed to be heavily degraded due to a lack of waterproofing and poor concrete quality. Abutment walls were exposed to water draining from above (Figure 10b). There was no visible damage due to the earthquake. No new cracks or movements of the substructure or superstructure elements were detected. No restrictions regarding traffic were given. Further inspection and maintenance were recommended due to the noticeable water leakage due to failed waterproofing and drainage.

Figure 10. Maja bridge: (**a**) side view of the bridge; (**b**) abutment wall.

4.5. Svračica Bridge

Svračica bridge is the furthest south bridge from town Glina that was examined (Figure 5). Its superstructure is a series of two continuous composite girders with over four even spans of 7.7 m (Figure 6). Dilatation is in the middle of the bridge. The superstructure comprises a multi-girder composite cross-section with 7 I280 steel girders placed 0.7 m apart, and a 20 cm thick reinforced concrete deck slab. The superstructure is 5.45 m wide in total, with 4.05 m wide roadway and asymmetric footways of 0.55 and 0.85 m partly supported by deck consoles (Figure 6). Columns are massive, 4 m high reinforced concrete structures with a variable cross-section ranging from 6 $\times$ 1 m at the bottom to 5.8 $\times$ 0.6 m at the top. All columns have a joint foundation slab that covers the whole riverbed (two spans), approximately 18 m long and 10 m wide. Abutments are minimal structures, about 2 m high and 4 m long with variable width and skewed abutment wings due to a road junction located immediately at their end. The superstructure is directly supported on the substructure elements without bearings. The bridge was in fairly good structural condition, showing moderate signs of steel girder corrosion, mostly in the vicinity of their supports (abutments) due to the longitudinal displacements at the ends of the bridge not being properly managed (no bearings) and water leakage from behind the abutment wall (Figure 11a). Since expansions joints are not present on the bridge, cracks were visible in the asphalt at the end of the superstructure. The deck slab concrete was in good condition. The concrete columns showed signs of reinforcement corrosion, with protective layer only locally spalling. The main reason for this problem is heavy water leakage due to non-existent drainage and waterproofing. Since there was a dilatation in the superstructure above the central column, the water is draining through the asphalt directly onto the column. The spalling of concrete showed a different type of concrete underneath and occurred at the contact of these different materials, so it very likely that columns had undergone rehabilitation work in the past. There was no visible damage from the earthquake, either in permanent deformations or movements of the superstructure or substructure elements. Therefore, the bridge was maintained for operation without restrictions. Further inspection and maintenance were recommended due to steel girder corrosion and signs of column reinforcement corrosion.

Figure 11. Svračica bridge main girder support: (**a**) abutment; (**b**) column.

4.6. Nikola Tesla Street Bridge

Nikola Tesla street bridge is located in the northeast part of town (Figure 5). It is a three span continuous girder bridge of 9.2 + 10 + 9.2 m (Figures 6 and 12a). Its superstructure comprises four I280 steel girders and two 52 cm high concrete ribs that are concreted in between two pairs of steel girders (Figure 6). Steel girders are thus partly concreted inside these ribs, also serving as a side formwork for concrete. The width of the concrete ribs is 1.0 m, and the inside distance between the ribs is 1.3 m. The width of the superstructure is 4.1 m, with an asymmetrical traffic area and footway only on one side. The superstructure is directly supported by columns and abutments. Columns are 5.2 m high, 4.3–4.6 m wide in the transverse, and 0.8–1 m wide in the longitudinal direction.

Figure 12. Nikola Tesla street bridge: (**a**) view of the bridge; (**b**) main girders to abutment support; (**c**) superstructure under view.

Abutment walls are 1.4 m high, and abutment wings are 2.9 m long. The bridge superstructure was found to be in a moderate-to-poor condition, with problems regarding structural steel corrosion and water leakage due to non-existent waterproofing and drainage. The asphalt layer was heavily worn out and almost completely missing in the footway area. The edges of superstructure consoles were missing a cornice, and reinforcement bars were protruding out of the concrete. Columns and abutments were found to be in fairly good condition, with no visible cracks or concrete spalling. Discoloration was visible due to water leakage from the superstructure onto columns, thus causing long-term damage to the column concrete and possibly reinforcement corrosion (Figure 12c). The bridge showed no signs of serious damage caused by the earthquake on the superstructure, substructure, or embankment slopes around abutments. Visible cracks were detected in the area of connection between the superstructure and abutment, where the superstructure is supported on the abutments, between the abutment wall and the cross girder (Figure 12b). Since this connection was not assumed as fixed in the statical system, it was expected that the opening of this crack occurred and was of no importance regarding bearing capacity. After inspection, the bridge was maintained for operation without restrictions. Further inspection and maintenance were recommended due to the poor state of the traffic surface (asphalt layer and footways), column reinforcement corrosion, failed waterproofing, and possible scour developing on column foundations.

4.7. Prekopa Bridge

Prekopa bridge crosses the river Maja at the north access road to Glina (Figure 5). The bridge has a continuous girder slab superstructure with over three spans of 10.15 + 12.55 + 10.15 m (Figure 6). Its reinforced concrete slab is 50 cm thick and 8.1 m wide, with 0.9 m consoles on each side. Its superstructure is supported by twin 3.9 m high columns at each side of the riverbed (the cross-section of each column is 1.0 × 0.5 m) and 2 m high and 1.7 m long abutments. There are no bearings on the bridge, and supports are realized as concrete hinged sections at the top of the substructure elements (Figure 13a). In comparison to the other examined bridges, this bridge was found to be fairly new, erected in 1999. An open drainage system and waterproofing were observed, so no serious long-term water damage was found.

Figure 13. Prekopa bridge: (**a**) side view of the bridge; (**b**) expansion joint.

Only traces of water leakage were visible on the abutment wall, probably due to the failed waterproofing of expansion joints at the ends of the bridge. There were hints of reinforcement corrosion on the abutment wall due to this water leakage. Expansion joints were also found to be clogged with dirt and gravel, with visible cracks in the asphalt layer around them (Figure 13b). The bridge was reported to be in very good condition; the superstructure and substructure concrete showed no signs of degradation or reinforcement corrosion. Except for expansion joints needing maintenance, no other notable problems were found. Due to it dating from a newer generation of bridges, it was certainly designed with seismic loads and seismic detailing, so no earthquake damage was expected nor found. This bridge performed exceptionally in this seismic event.

4.8. Hađer Bridge

Hađer bridge is just north of town Glina, leading to a nearby settlements west of the river Glina (Figure 5). It crosses river Glina just at the mouth of river Maja. It is the longest of all the bridges, with relatively large spans and tallest columns, so the seismic action was certainly the highest here. Being built in 1987, it was presumably designed with a certain degree of seismic behavior in mind. The bridge comprises a series of one continuous slab girder with over two spans of 19.95 + 20.6 m and one simply supported girder spanning 19.95 m (Figure 6). Between these girders, there is a visible dilatation above one pier (Figure 14d). The superstructure comprises three hollow girders (each girder with a 210 × 75 cm cross-section) connected by longitudinal in situ concrete joints and in situ 20 cm thick concrete deck plate above them. The deck plate continues to consoles at each side of the cross-section to support footways. This type of cross-section can be regarded as a slab cross-section. The overall width of the superstructure is 8.6 m. The substructure comprises two single circular cross-section columns with a wide consoled head cross girder to accommodate the supports of each girder. The abutments are massive reinforced concrete structures. The bridge has no drainage system, no bearings, and no expansion joints, all of which resulted in durability problems and limited displacement capacity (as is elaborated later). The bridge showed signs of heavy water leakage from the superstructure on abutment walls, causing concrete degradation and spalling, and reinforcement corrosion on both the superstructure and abutment (Figure 14c). The same problem was present at the dilatation above the central column, where the water is leaking through the dilatation and causing damage to the ends of the girder slabs. The head of the column, as well as the abutment wall and wing console (Figure 14c,d), already showed progressive reinforcement corrosion, with parts of the concrete protective layer missing due to delamination. There were wide visible cracks in the column head girder and visible reinforcement bars due to corrosion (Figure 14d). It was also noticeable that the girders were not symmetrically supported on the column head, i.e., one girder was found to have a longer support length than the other, which was not correctly executed in the erection process. This poses a potential danger in an event of even stronger earthquake since inadequate support length could cause the girder to slip from the column head. No horizontal seismic limiting element was found, so only support length insured this from happening.

Earthquake-related damage was recorded on several elements of the bridge. It was evident that the bridge superstructure moved both longitudinally and transversely at a notable rate. The first proof of such movements could be seen on the asphalt layer at the ends of the bridge superstructure, which was heavily cracked, waved, and delaminated upwards (Figure 14a). This movement also caused the fracture of bridge cornice, which was executed without any dilatation between the abutment and superstructure (Figure 14c,e). It is surprising that a bridge this long does not have expansion joints at the ends that would accommodate for such movements without causing any damage. Secondly, transverse movements caused heavy damage to abutment side walls, thus cracking and completely fracturing parts of them (Figure 14b). It is also incorrect that the abutment side wall is so close to the superstructure with no tolerances for transverse movements of any degree (the abutment was found to be too narrow for this width of the superstructure). The structure of this bridge is very flexible due to tall single piers of circular cross-section, positioned centrally along bridge axis. Therefore, it is not surprising that the bridge exhibited large movements during the earthquake; it is more surprising that these movements were not accounted for in the design of both the expansion joints and spacing tolerances between the substructure and superstructure elements.

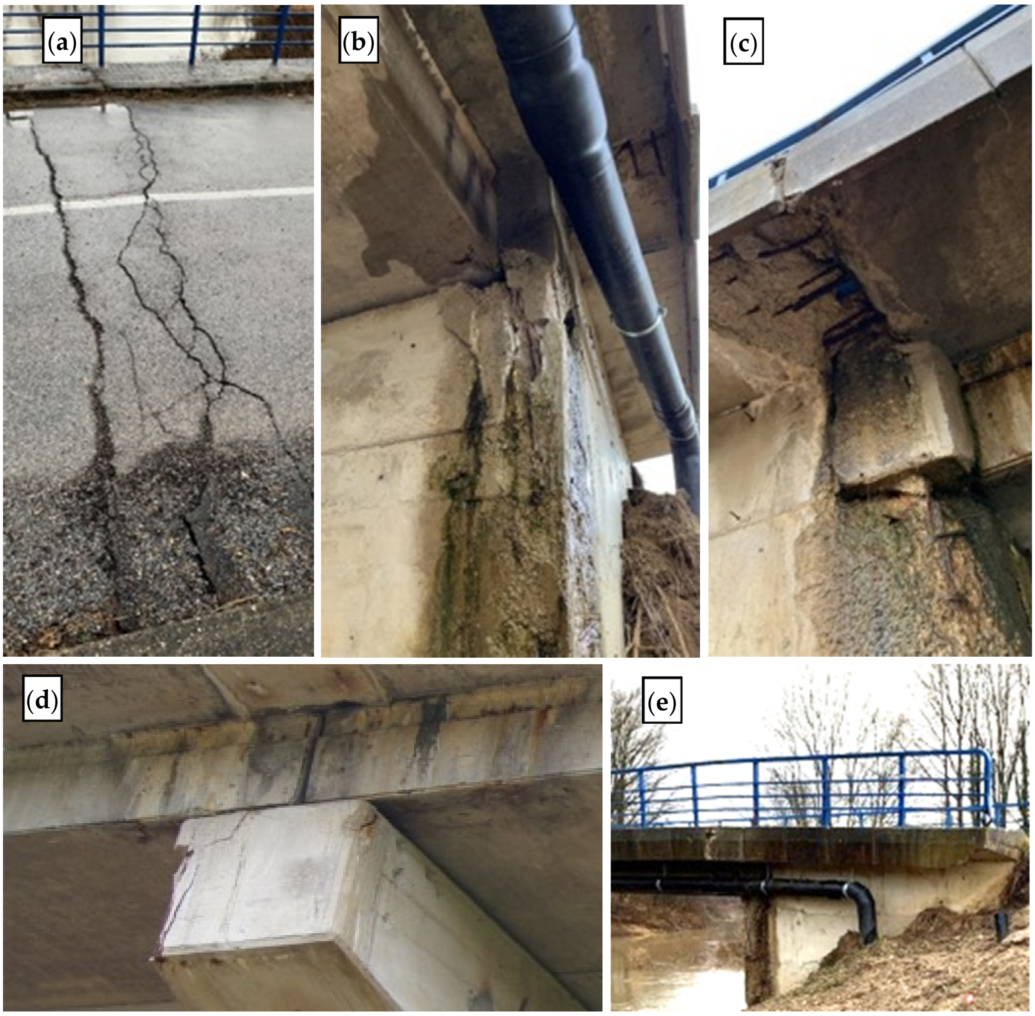

Figure 14. Hađer bridge: (**a**) damage caused by earthquake in the asphalt; (**b**) fractured abutment side wall due to the earthquake; (**c**) water leakage from superstructure end, causing damage to abutment; (**d**) uneven support length for the girders; (**e**) cracked cornice due to earthquake movement.

It is evident that all the damage due to the earthquake could have been easily avoided if proper design rules had been followed. Besides the described damage, which can be regarded as non-structural, the bridge performed well in this earthquake event. Further inspection and rehabilitation were recommended due to aforementioned earthquake damage and heavy reinforcement corrosion of column heads and abutment walls.

5. Immediate Strengthening Measures

Only one of the assessed bridges needed closure and immediate strengthening measures. As discussed in Section 4.2, Matija Gubec Street bridge leading to Majske Poljane showed signs of west abutment slippage and rotation, and its stability and integrity were compromised. Following the closure of the bridge for traffic, emergency measures were prescribed for the immediate strengthening of the abutment to prevent its total collapse. The immediate danger of collapse was even more emphasized due to heavy rains that followed in the days after the earthquake, which caused the water level to rise and further erode the west riverbank that had already shown scour signs under the abutment foundation. Additionally, multiple aftershocks threatened the unstable balance of the abutment wall. The decision was made to use large stone block material and to fill the slope of the riverbank up to the top of the abutment, around its wings, and as far as the middle columns in the riverbed (Figure 15a). The stone material needed to be of very large fractions (from 50 to 100 cm) to prevent it from being washed away by the river flow. The abutment foundation, wall, and wings were enveloped by this stone infill, thus preventing the further erosion of the soil and acting as a support for the abutment (Figure 15b). The whole work was performed over just two days in hard working conditions due to soaked soil from continuous rains. The measure was proven to be effective since it stopped the further movement of the abutment. Nevertheless, the bridge was closed for traffic as a precautionary measure until it was to be evaluated further and permanent solutions were found.

Figure 15. Immediate measures to secure the abutment of Matija Gubec street bridge: (**a**) beginning of the work; (**b**) completed infill.

6. Recommendations for Further Rehabilitation Work and Current Progress

Following the detailed evaluation and assessment of the previously strengthened bridge, it was concluded that extensive rehabilitation work was needed. The bridge stone wall abutment was deemed unsalvageable and to be replaced with new reinforced concrete abutment. The new reinforced concrete abutment will have a deeper and wider foundations, with the stone and shotcrete cladding of the embankment slope and river bank to prevent scour. One of the columns that showed scour signs and had an insufficient foundation-bearing capacity is also to be replaced. The new reinforced concrete column will have an 80×400 cm cross-section and a 300×550 cm, 100 cm thick foundation slab. This foundation will also be protected by stone and shotcrete cladding. One column and east abutment are to be salvaged and jacketed with a new 10 cm thick layer of reinforced concrete. Since the superstructure was heavily degraded, with progressive structural steel and reinforcement corrosion, missing parts of the footways, and low remaining load-bearing capacity, it was decided that it also needs to be replaced. The new 50 cm thick reinforced concrete slab superstructure will also be wider (600 cm), accommodating more traffic width for vehicles and pedestrians. The superstructure and substructure elements will be integrally connected without any bearings to achieve better durability. The bridge will be equipped with waterproofing and closed drainage system. Figure 16 shows the current progress of this rehabilitation.

Figure 16. Progress of rehabilitation work of Matija Gubec street bridge.

Regarding the recommendations for the other examined bridges, the Hađer Bridge most critically needs detailed inspection and rehabilitation following earthquake damage. Since this damage was caused by improper movement management, it is recommended that expansion joints allowing for seismic movements are added at the bridge ends and above the central dilatation column. The inelastic movements should also be checked by nonlinear calculations to determine if the safe tolerances against slippage of the girders are met.

An extensive detailed inspection and NDT due to durability issues were recommended for Roviška bridge, Maja bridge, Svračica bridge, Nikola Tesla Street bridge, and Hađer bridge, as previously stated in the corresponding sections.

Thus far, only the rehabilitation of Matija Gubec Street bridge has been undertaken.

7. Conclusions

On 29 December 2020, a devastating $M_L = 6.2$ earthquake hit the Sisak-Moslavina county of Croatia. Immediately after the earthquake, structural engineers' teams were dispatched to conduct rapid damage assessment and evaluate the usability of structures. Eight evaluated bridges located in Glina county have been discussed in this paper as case studies. Only one bridge with major damage was closed for traffic, and others were opened for continued use without restrictions. Most of the bridges performed well in the earthquake (Table 1), with major damage attributed to Matija Gubec Street bridge and minor damage attributed to Hađer bridge. Seismic retrofitting is recommended for both bridges. For the former, this retrofitting has already been undertaken, and half of the substructure and the whole superstructure will be replaced. For the second bridge with minor damage, it has been recommended to add retrofitting measures to allow for seismic superstructure horizontal movements and prevent excess movement that could result in catastrophic failure.

Table 1. Overview of seismic damage, degradation, and design flaws of examined bridges.

Bridge	Seismic Damage	Degradation	Design Flaws
Glina bridge	No damage	Expansion joints clogged, steel corrosion	Not evident
Matija Gubec Street bridge	Abutment sliding and block wall damage	Heavy steel corrosion, concrete spalling and reinforcement corrosion, scour, footways and cornice partly missing	Insufficient foundation for one column, one abutment from stone blocks and one from reinforced concrete, no waterproofing and drainage
Roviška bridge	No damage	Abutment wall reinforcement corrosion, parts of side wall missing, heavily degraded footway and cornice, possible column foundation scour	Abutment side wall with insufficient reinforcement and poor concrete, no waterproofing and drainage
Maja bridge	No damage	Clogged drainage, heavily degraded footway, abutments exposed to water damage	Poor waterproofing, dilatation between abutment wall and wings
Svračica bridge	No damage	Steel girder corrosion, columns reinforcement corrosion	No waterproofing or drainage
Nikola Tesla Street bridge	No damage	Steel girder corrosion, columns reinforcement corrosion, heavily degraded footway and cornice, heavy asphalt damage	No waterproofing or drainage
Prekopa bridge	No damage	Expansion joint clogged and with failed waterproofing, abutment wall water leakage	Not evident
Hađer bridge	Excessive movements, abutment wall damage, asphalt damage, cornice damage	Heavy water leakage on abutment walls and column head, concrete spalling and reinforcement corrosion, cracked column head	No expansion joints, no bearings, no drainage, girders not symmetrically supported

As a benefit of these examinations, many other durability-related problems and design flaws were also discovered (Table 1), demonstrating that the progressive deterioration of materials and elements had already started. All the examined bridges were lacking in regular maintenance or even periodical inspection to such a degree that rehabilitation work has been recommended for some. The common deficits observed for all the bridges are as follows: a lack of superstructure waterproofing, non-existent or failed drainage, the corrosion of reinforcement and/or structural steel, footways and consoles with missing parts of concrete and cornice, the cloggage of expansion joints (when they are present), and damage to the asphalt layer.

However, despite long service lives and insufficient maintenance, most of the bridges performed well during this earthquake event and continued to be used after the earthquake for rescue and evacuation purposes.

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References

1. Government of Croatia World Bank Report. *Croatia Earthquake Rapid Damage and Needs Assessment*; Government of Croatia World Bank Report: Zagreb, Croatia, 2020.
2. Perković, N.; Stepinac, M.; Rajčić, V.; Barbalić, J. Assessment of Timber Roof Structures before and after Earthquakes. *Buildings* **2021**, *11*, 528. [CrossRef]
3. Stepinac, M.; Kisicek, T.; Renić, T.; Hafner, I.; Bedon, C. Methods for the Assessment of Critical Properties in Existing Masonry Structures under Seismic Loads-the ARES Project. *Appl. Sci.* **2020**, *10*, 1576. [CrossRef]
4. Novak, M.Š.; Uroš, M.; Atalić, J.; Herak, M.; Demšić, M.; Baniček, M.; Lazarević, D.; Bijelić, N.; Crnogorac, M.; Todorić, M. Zagreb Earthquake of 22 March 2020—Preliminary Report on Seismologic Aspects and Damage to Buildings. *Gradjevinar* **2020**, *72*, 843–867. [CrossRef]
5. Markušić, S.; Stanko, D.; Penava, D.; Ivančić, I.; Oršulić, O.B.; Korbar, T.; Sarhosis, V. Destructive M6.2 Petrinja Earthquake (Croatia) in 2020—Preliminary Multidisciplinary Research. *Remote Sens.* **2021**, *13*, 1095. [CrossRef]
6. Korbar, T.; Markušić, S.; Stanko, D.; Penava, D. Petrinja M6.2 earthquake in 2020 damaged also solid linear: Are there similar active faults in Croatia? In Proceedings of the 1st Croatian Conference on Earthquake Engineering 1CroCEE, Zagreb, Croatia, 22 March 2021; Faculty of Civil Engineering, University of Zagreb: Zagreb, Croatia, 2021; pp. 353–362.
7. Bacic, M.; Sasa Kovacevic, M.; Librić, L.; Žužul, P. Sinkholes induced by the Petrinja M6.2 earthquake and guidelines for their remediation. In Proceedings of the 1st Croatian Conference on Earthquake Engineering 1CroCEE, Zagreb, Croatia, 22 March 2021; Faculty of Civil Engineering, University of Zagreb: Zagreb, Croatia, 2021; pp. 341–352.
8. Grünthal, G.; European Seismological Commission; Working Group "Macroseismic Scales". *European Macroseismic Scale 1998: EMS-98*; European Seismological Commission, Subcommission on Engineering Seismology, Working Group Macroseismic scales: Luxembourg, 1998.
9. Spence, R.; Foulser-Piggott, R. The international macroseismic scale-extending EMS-98 for global application. In Proceedings of the Second European Conference on Earthquake Engineering and Seismology, Istambul, Turkey, 25 August 2014.
10. Milic, I.; Ivankovic, A.M.; Syrkov, A.; Skokandic, D. Bridge Failures, Forensic Structural Engineering and Recommendations for Design of Robust Structures. *Gradjevinar* **2021**, *73*, 717–736.
11. Yokoyama, K.; Inaba, N.; Honma, A.; Ogata, N. Development of Bridge Management System for Expressway Bridges in Japan. *Tech. Memo. Public Work. Res. Inst.* **2006**, *4009*, 99–104.
12. Goran Puž, A.; Radić, J.; Puž, G.; Tenžera, D. Bridge Condition Forecasting for Maintenance Optimisation. *Građevinar* **2013**, *65*, 1079–1088. [CrossRef]
13. Estes, A.C.; Asce, M.; Frangopol, D.M.; Asce, F. Updating Bridge Reliability Based on Bridge Management Systems Visual Inspection Results. *J. Bridge Eng.* **2003**, *8*, 374–382. [CrossRef]
14. Tenžera, D.; Puž, G.; Radić, J. Visual Inspection in evaluation of Bridge Condition. *Gradjevinar* **2012**, *64*, 717–726.
15. Gattulli, V.; Chiaramonte, L. Condition Assesment by Visual Inspection for a Bridge Management System. *Comput.-Aided Civ. Infrastruct. Eng.* **2005**, *20*, 95–107. [CrossRef]
16. Quirk, L.; Matos, J.; Murphy, J.; Pakrashi, V. Visual Inspection and Bridge Management. *Struct. Infrastruct. Eng.* **2018**, *14*, 320–332. [CrossRef]
17. Graybeal, B.A.; Phares, B.M.; Rolander, D.D.; Moore, M.; Washer, G. Visual Inspection of Highway Bridges. *J. Nondestruct. Eval.* **2002**, *21*, 67–83. [CrossRef]
18. Agdas, D.; Rice, J.A.; Martinez, J.R.; Lasa, I.R. Comparison of Visual Inspection and Structural-Health Monitoring as Bridge Condition Assessment Methods. *J. Perform. Constr. Facil.* **2016**, *30*, 04015049. [CrossRef]
19. Marić, M.K.; Ivanković, A.M.; Vlašić, A.; Bleiziffer, J.; Srbić, M.; Skokandić, D. Assessment of Reinforcement Corrosion and Concrete Damage on Bridges Using Non-Destructive Testing. *Gradjevinar* **2019**, *71*, 843–862.
20. Ivanković, A.M.; Skokandić, D.; Marić, M.K.; Srbić, M. Performance-Based Ranking of Existing Road Bridges. *Appl. Sci.* **2021**, *11*, 4398. [CrossRef]
21. Mandić Ivanković, A.; Skokandić, D.; Žnidarič, A.; Kreslin, M. Bridge Performance Indicators Based on Traffic Load Monitoring. *Struct. Infrastruct. Eng.* **2019**, *15*, 899–911. [CrossRef]
22. Skokandić, D.; Mandić Ivanković, A. Value of Additional Traffic Data in the Context of Bridge Service-Life Management. *Struct. Infrastruct. Eng.* **2020**, 1–20. [CrossRef]
23. Pelà, L.; Aprile, A.; Benedetti, A. Seismic Assessment of Masonry Arch Bridges. *Eng. Struct.* **2009**, *31*, 1777–1788. [CrossRef]
24. Pelà, L.; Aprile, A.; Benedetti, A. Comparison of Seismic Assessment Procedures for Masonry Arch Bridges. *Constr. Build. Mater.* **2013**, *38*, 381–394. [CrossRef]
25. Franetović, M.; Ivanković, A.M.; Radić, J. Seismic Assessment of Existing Reinforced-Concrete Arch Bridges. *Gradjevinar* **2014**, *66*, 691–703. [CrossRef]
26. Monteiro, R.; Delgado, R.; Pinho, R. Probabilistic Seismic Assessment of RC Bridges: Part I—Uncertainty Models. *Structures* **2016**, *5*, 258–273. [CrossRef]
27. Monteiro, R. Sampling Based Numerical Seismic Assessment of Continuous Span RC Bridges. *Eng. Struct.* **2016**, *118*, 407–420. [CrossRef]
28. Cassese, P.; de Risi, M.T.; Verderame, G.M. Seismic Assessment of Existing Hollow Circular Reinforced Concrete Bridge Piers. *J. Earthq. Eng.* **2020**, *24*, 1566–1601. [CrossRef]

29. Srbić, M.; Ivanković, A.M.; Vlašić, A.; Kovačević, G.H. Plastic Joints in Bridge Columns of Atypical Cross-sections with Smooth Reinforcement without Seismic Details. *Appl. Sci.* **2021**, *11*, 2658. [CrossRef]
30. Homaei, F.; Yazdani, M. The Probabilistic Seismic Assessment of Aged Concrete Arch Bridges: The Role of Soil-Structure Interaction. *Structures* **2020**, *28*, 894–904. [CrossRef]
31. Zelaschi, C.; Monteiro, R.; Pinho, R. Parametric Characterization of RC Bridges for Seismic Assessment Purposes. *Structures* **2016**, *7*, 14–24. [CrossRef]
32. Morbin, R.; Zanini, M.A.; Pellegrino, C.; Zhang, H.; Modena, C. A Probabilistic Strategy for Seismic Assessment and FRP Retrofitting of Existing Bridges. *Bull. Earthq. Eng.* **2015**, *13*, 2411–2428. [CrossRef]
33. Stepinac, M.; Lourenço, P.B.; Atalić, J.; Kišiček, T.; Uroš, M.; Baniček, M.; Šavor Novak, M. Damage Classification of Residential Buildings in Historical Downtown after the ML5.5 Earthquake in Zagreb, Croatia in 2020. *Int. J. Disaster Risk Reduct.* **2021**, *56*, 102140. [CrossRef]
34. Lulić, L.; Ožić, K.; Kišiček, T.; Hafner, I.; Stepinac, M. Post-Earthquake Damage Assessment-Case Study of the Educational Building after the Zagreb Earthquake. *Sustainability* **2021**, *13*, 6353. [CrossRef]
35. Baggio, C.; Bernardini, A.; Colozza, R.; Pinto, A.; Taucer, F. *Field Manual for Post-Earthquake Damage and Safety Assessment and Short Term Countermeasures (AeDES)*; Translation from Italian: Maria ROTA and Agostino GORETTI; European Commission: Luxembourg, 2007.
36. Setunge, S.; Li, C.Q.; Mcevoy, D.; Zhang, K.; Mullett, J.; Mohseni, H.; Mendis, P.; Ngo, T.; Herath, N.; Karunasena, K.; et al. *Failure Mechanisms of Bridge Structures under Natural Hazards*; RMIT: Melbourne, Australia, 2018.
37. Tanimura, S.; Sato, T.; Umeda, T.; Mimura, K.; Yoshikawa, O. A Note on Dynamic Fracture of the Bridge Bearing Due to the Great Hanshin-Awaji Earthquake. *Int. J. Impact Eng.* **2002**, *27*, 153–160. [CrossRef]
38. Mitchell, D.; Huffman, S.; Tremblay, R.; Saatcioglu, M.; Palermo, D.; Tinawi, R.; Lau, D. Damage to Bridges Due to the 27 February 2010 Chile Earthquake. *Can. J. Civ. Eng.* **2013**, *40*, 675–692. [CrossRef]
39. Kawashima, K.; Buckle, I. Structural Performance of Bridges in the Tohoku-Oki Earthquake. *Earthq. Spectra* **2013**, *29*, 315–338. [CrossRef]
40. Hajdin, R.; Kušar, M.; Mašović, S.; Linneberg, P.; Amado, J.; Tanasić, N. *COST Action TU1406: Quality Specifications for Roadway Bridges, Standardization at a European Level*; WG3 Technical Report: Establishment of a Quality Control Plan; COST: Brussels, Belgium, 2018.