Incorporating Local Environmental Factors into Railway Bridge Asset Management

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Abstract

A novel approach to comparing bridge deterioration rates under different environmental conditions is employed using a network analysis approach. This approach uses a matrix condition scoring system utilised by Network Rail (NR). It does not require any conversion factors which can introduce subjectivity. A number of different factors were analysed to ascertain if they have an effect on bridge deterioration. The key factors were identified and their deterioration profiles incorporated into a probabilistic Petri-Net (PN) model, calibrated with historical data. From these, comparative model outputs pinpointing which factors affect bridge deterioration the most can be computed. Finally, simulations were carried out on the PN model to evaluate which of the factors would have the most financial effect for a transport agency. This allows a bridge manager to categorize bridges in different deterioration sets allowing the definition of different optimal inspection and maintenance strategies for each set.

Keywords: Rail, Bridges, Asset Management, Maintainability, Environmental Factors, Petri-Net

1. Introduction

The railway is intrinsic to the UK transport sector, however much of the infrastructure is aged and requires regular maintenance. This maintenance, in turn, means that railway managers incur huge costs to maintain their portfolios of infrastructure. Railway structures are critical to the smooth operation of the system. Over time more and more demand has been placed on these structures; with the advancement of more sophisticated signalling systems, trains can run closer together and so the demand increases ever more. This means an effective understanding of the assets is critical to be able to manage them successfully. There has been many studies involving bridge deterioration modelling using a number of different techniques (Morcous et al., 2010; Frangopol et al., 2001; Arditi and Tokdemir, 1999; Neves and Frangopol, 2005). Many of these authors argue that there are a number of external factors that also affect bridge deterioration, but there are few studies which try to ascertain what the factors are, how they affect the deterioration profile and how much their influence would cost to a railway structure portfolio manager.

Network Rail (NR) data has been used to conduct this research. NR is the largest railway infrastructure manager in the UK. However, the results presented in this study have a wide interest to any organisation responsible for these type of structures.

2. Literature Review

Many studies involving bridge deterioration models hint towards the fact that there may be a multitude of different external factors affecting bridge deterioration. The problem with identifying these factors is that a significant pool of data is required to be able to conclude that there is a difference in deterioration profiles. The factors commonly considered include: asset age, traffic volume, span length, number of tracks, structure type, coastal proximity and temperature variation (Morcous et al., 2002; Niroshan et al., 2015; Patjawit and Kanok-Nukulchai, 2005).

Jiang and Sinha (1989) and Jiang (1990) conducted a study in which 5,700 highway bridges in Indiana, USA, were analysed. They created a Markov based model and analysed a number of different categories to understand the effect of different bridge attributes. For instance, the bridges were split into which highway system they were part of: "interstate highways bridges" or "other state highway bridges"; by traffic volume, interstate highways bridges was split between those which experienced less than 10,000 Average Daily Traffic (ADT) and more than 10,000 ADT; other state highways the data was split by those which experience less than 5,000 ADT, 5,000 to 10,000 ADT and those which experience more than 10,000 ADT; the climatic regions were analysed, split up into the bridges which were Northern or Southern. For each of the factors a sample set of 50 bridges was selected and used to see if the particular bridge category experienced different deterioration to the rest of the population. The results indicated that for most of the bridge subgroups, the factors being analysed were not significant enough to deviate from the standard deterioration experienced by the rest of the population. However, the results did find a statistical

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difference between the deterioration of interstate highways bridges and other state highway bridges. The authors concluded the analysis of the factors by stating that they will provide different performance curves for interstate highways bridges and other state highway bridges.

Scherer and Glagola (1994) studied the highway bridges in Virginia, USA. The authors explain that there are 13,000 bridges in the district and they attempt to model the deterioration process using a Markov approach. Seven condition states were selected and the authors explain that due to the Markov state-space explosion characteristic, the number of states would have been $7^{13,000}$ which was infeasibly large to be calculated by contemporary computing power. So a process of grouping was performed where similar bridges were pooled together to reduce the number of states in the final model. In total 216 cases were created which reduced the number of states to 7^{216}. The groups were decided upon a number of factors including the structure type, road network, environmental condition, age of the structure and the traffic loading. The bridges were split up into which road network they were part of: interstate, urban extension and secondary. In terms of climate the data was split into: East coast, Piedmont plateau and the Western mountains. Finally, the traffic loading was split into a number of different ADT values depending on the size of the road network, ranging from 500 ADT for bridges on secondary road networks to greater than 5,000 for bridges on interstate road networks. The authors then explain the assumption that the bridges are grouped into the appropriate categories and that similar bridges in analogous conditions will have comparable performance and deterioration characteristics.

Agrawal et al. (2010) also uses the 7 condition states used by Scherer and Glagola (1994). The authors describe an approach to provide the probabilistic lifetime distributions using the Weibull distribution of 17,000 highway bridges across New York State, USA. They use case study elements to study the effects of external factors on the lifetime of the element. The examples used in the study related to the type of materials used in construction. An analysis was performed to see whether girders manufactured from steel deteriorate faster or slower than those made from weathering steel. The result of the analysis showed that elements made from weathering steel deteriorate at the same rate as standard steel elements for the first 20 years of the elements life. Beyond that point weathering steel seems to degrade slower than standard steel elements. The authors continue to analyse the difference in deterioration rates between structures with epoxy coated reinforcement bars and uncoated reinforcement bars. Analysis of the factors that affect deterioration are key to this study, however the focus remains on construction materials rather than external environmental factors.

Huang et al. (2010) provides a useful summary regarding all the studies in literature carried out on this topic. They state that there are a plethora of weathering factors that affect bridge deterioration. The study used inspections from 2,128 bridges in Taiwan, including traffic and weather data. The study looks at the most common types of defects for reinforced concrete and then tries to ascertain what the major and minor causes of that defect are. For instance the author concludes that the traffic volume is a major factor in the corrosion of the reinforcement bars. However, distance from the coast is a factor for both spalling and fragmentation. The author then groups the factors by the defect they are likely to cause; cracking seems to be the most sensitive defect as 8 of the 10 factors affect it, however honeycombing is only affected by two factors: the peak monthly rainfall and the maximum days of rain in the month. The author has calculated that distance from the coast is one of the factors that affects bridge deterioration, but it was not one of the major factors. However, the author also states that their sample of bridges did not contain any that could be considered coastal.

Zhao and Chen (2002) performed a study regarding the causes of structural deterioration using a fuzzy logic system. A case study exercise was performed where the most critical bridge defects, cracking and spalling, were selected. The artificial intelligence system was used to find the causes of the defects. The parameters included: the structure type, bridge age and overall span length amongst others. The results suggested that cracking was highly dependant on the loading caused by traffic but the construction technique and structural design had little effect. However for spalling, the "other" factors (e.g. bridge age) had the most effect whereas loading was much less of a factor in deterioration.

In summary, a variety of literature has been evaluated. Each study used its own approach of identifying the external factors and assessing their effects. The over-riding conclusion that can be drawn is that there are a variety of external factors and they can greatly affect bridge deterioration.

3. Condition States, Deterioration and Maintenance Policies

3.1. Condition States

Structures are inspected and its condition is recorded according on the Severity Extent Rating (SevEx). The rating system is alphanumeric containing both the classification and intensity of the defect and its extent. The SevEx conditions vary depending on the superstructure material. For concrete structures, the condition ratings go from A1, a new structure, to G6 a heavily deteriorated structure. According to available inspection data, the most important damage for concrete structures is either spalling or cracking. Nielsen et al. (2013) found the percentage of concrete structures suffering from either spalling or cracking can reach 89.9%. The full list of SevEx condition ratings can be found in Table 1.

Table 1: SevEx defects for concrete structures (Network Rail, 2012b)

Severity	Defect Definition
A	No visible defects
B	Surface damage, Minor spalling, Wetness, Staining, Cracking <1mm wide
C	Spalling without evidence of corrosion, Cracking ≥ 1mm wide without evidence of corrosion
D	Spalling with evidence of corrosion, Cracking ≥ 1mm wide with evidence of corrosion
E	Secondary reinforcement exposed
F	Primary reinforcement exposed
G	Structural damage to element including permanent distortion

Extent	Definition
1	No visible defects
2	Localised defect due to local circumstances.
3	Affects <5% of the surface of the element.
4	Affects 5%-10% of the surface of the element.
5	Affects 10%-50% of the surface of the element.
6	Affects >50% of the surface of the element.

3.2. Maintenance Actions

A system of conversion is used to calculate which maintenance action is most appropriate for the defect being identified (Network Rail, 2012a). During inspection, the SevEx condition rating system is used. During processing, this is converted to a numerical score known as Structure Condition Marking Index (SCMI). A SevEx condition rating A1 would translate to an SCMI score of 100; conversely, a G6 condition score would translate to an SCMI score of 0. The SCMI score is used with the policy documents to ascertain which maintenance action is the most appropriate for that particular element condition. Elements should not breach the "Basic Safety Limit" beyond which urgent replacement of the element is carried out. The limit is dependant on the superstructure material type.

For the present model, this process was back-converted. Rather than converting from SevEx to SCMI and then finding the correct maintenance action, there was a conversion straight from SevEx to the corresponding maintenance action. This back-conversion allowed the maintenance actions to be directly incorporated into the model which means that maintenance decisions could be made dynamically during simulation.

There seems to be a discrepancy between the policy document (Network Rail, 2012b) and the actual work carried out on the structures. Although there is a threshold that the condition must breach before being maintained, there were many cases where elements were maintained above this threshold. Upon consultation, it seems that there are, in fact, three types of maintenance. That which is carried out beyond the maintenance threshold, known forth as Major Repair. Replacements, which are carried out beyond the "Basic Safety Limit". And those which

are performed above the threshold for repair, known forth as Minor Repairs. Table 2 details the SevEx conditions relating to the corresponding maintenance action.

Table 2: The appropriate maintenance actions for different SevEx conditions.

SevEx States	Maintenance Action
B2-B4, C2-C3, D2	Minor Repair
B5-B6, C4-C6, D3-D5, E2-E4, F2-F3	Major Repair
D6, E5-E6, F4-F6, G2-G6	Replacement

3.3. Deterioration Calibration

To be able to compare different Local Environmental Factors (LEFs), a robust system of condition scoring must be used. Most models that study bridge deterioration modelling use a linear system of conditions (Cesare et al., 1992; Morcous, 2006; Le and Andrews, 2015) and so it is fairly straightforward to convert these into a numerical condition score which can then be compared against one another. E.g. an element that is in the Poor condition may attain a score of 40 whereas an element in the Good condition state may attain a score of 75 and so it can be said the condition of the element is almost twice as good as the element in the Poor condition.

The condition rating system used in this study is a 2-Dimensional scale. This means that the defect and the severity of the defect can evolve independently over time. The NR guidelines offer advice on how to convert from the 2-D SevEx system to a 1-D numerical SCMI score. However, this method uses factors to convert from one system to another which may not be reliable.

Another approach was to simply use the number of condition states between inspections as a rate of degradation e.g. an element degraded 4 condition states over 6 years and therefore degraded at a rate of 0.67 condition states/year. However, this method assumes that the difference between the states are constant and equal, which may not be an appropriate assumption. To go from condition state B2 to B3 means that the defect has increased in extent by 5%, however to go from B2 to C2 means that the defect has evolved from light cracking to moderate cracking. Developing a conversion matrix from the 2-D SevEx system into a numerical scoring system is difficult because weighting the conversion factors would involve expert judgement which always has a level of subjectivity.

An alternative approach has been developed which allows the states to remain in 2-D form and be independent of one another. No conversion factors are applied which means that there is no ambiguity about the weightings or conversion factors. The method involves using network analysis to calculate the shortest paths from one condition state to another. The distance between nodes is the Mean Time to Failure (MTTF). The higher the probability of an element taking a particular route, the lower the

MTTF value, and in turn, a higher failure rate. Therefore it was most appropriate to calculate the distances using the shortest path method as it coincides with the most likely used route. Dijkstra's algorithm (Dijkstra, 1959) was used to calculate the shortest path between nodes. The way the algorithm works is to begin at the start node and analyse the distances of all the immediately neighbouring nodes. The shortest distance is selected and then that node becomes the primary node. The primary node is checked for its distance to its immediately neighbouring nodes and the shortest one is selected as the new primary node. This process continues until the shortest path has been calculated from the chosen start and end nodes. In this study the shortest path will reveal the most likely route from one condition state to another and the shortest path MTTF value between the two states. Failure in this context is defined as reaching the destination node. An example of a network diagram is shown in Figure 1. A subset of conditions, A1 to D4 are shown, along with the possible paths and the path distances. The path distances are the MTTF times, calculated in years. The shortest path is shown in the solid line.
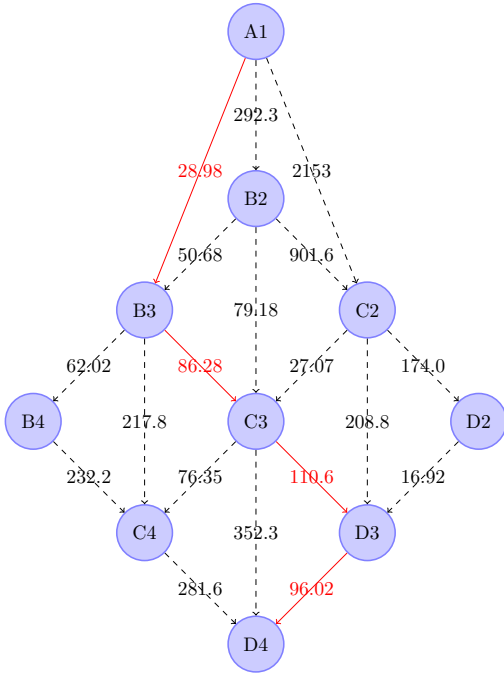

Figure 1: Network map showing a subset of the condition states and example MTTF times between each state. The shortest path from A1 to D4 is shown in the solid line.

4. Methodology

The network analysis approach has been used to find the MTTF times between various thresholds. Namely, the time it takes to get from a certain condition state to the Major Repair and Replacement thresholds. The Minor Repair threshold was not considered because it begins at

condition state B2 and therefore only state A1 would be considered. The thresholds are back-converted from Network Rail guidelines as shown in Table 2. The thresholds are simply the condition states where Major Repair or Replacement would be required. The MTTF times to get to these thresholds are used as a comparison to ascertain if the LEF is playing a factor in the degradation of the element.

The way this works in practice is to match the elements to its corresponding attribute e.g. its proximity to the coast. Then split the dataset according to the attribute in use e.g. elements within 20km of the coast. Once the data has been split into the number of categories required, the element inspections are processed into a matrix of occurrences. This provides an indication of how the defects evolve over time. From the occurrences, a matrix of MTTF times can be calculated. This can then be used with Dijkstra's algorithm to compute the shortest paths to the Major Repair and Replacement thresholds. This is then used in the results analysis to determine whether there is evidence of significantly different structural deterioration.

5. Data Source

A variety of different datasets were provided for this study: the Cost Analysis Framework (CAF), the Civil Asset Register and Reporting System (CARRS), MONITOR and the Structure Condition Monitoring Index (SCMI). CAF contains larger items of repair work, including details of the condition and the work required. CARRS contains general asset information for portfolio overviews, used mainly for system level observations. MONITOR is similar to CAF, however is used for smaller items of work carried out by NR maintenance teams. Finally SCMI contains the inspection records, including both the SevEx and SCMI scores. Each dataset is used for a different corporate function and so a process of cleaning and combining data was required so that the data was in the formats required for the study. The final combined dataset spanned from 1998 to 2014.

NR use an element hierarchy to organise their assets. The total number of inspected bridges is 25,949. Each asset is divided into a number of different Major Elements e.g. deck, end supports and mid supports. The Major Elements are element groups rather than distinct elements. There are 273,427 inspected Major Elements in the database. Each of the Major Elements are split up into Minor Elements e.g. bearings, girders and wing walls. There are 563,150 Minor Elements that have been inspected. In total there are 1,397,748 inspections on Minor Elements. These quantities have been structured in Figure 2.

Concrete girders are used in this study as the example element. This is because concrete structures are becoming more common and so their management is becoming more important. The number of concrete bridges in the database totals 4,434. These are the primary load bearing
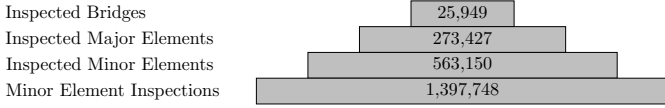
Figure 2: Element hierarchy and the associated quantity. Quantities calculated from the NR databases.

elements and so are critical to the health of the structure. The total number of inspections on concrete main girders is 407,708. The difference in condition between inspections was used to calibrate the deterioration profile of the element.

To be able to calibrate the deterioration profile, a change in the condition of the element is required. This means elements need to have at least two inspections to be used for deterioration calibration. The population of assets is vast and it takes a significant amount of time to inspect every one and so most of the elements that have had repeat inspections have only two inspections. The elements that have had two repeat inspections accounts for 82.68% of all elements. Some elements have had three repeat inspections (15.83%), presumably they are in regions that have enough resources to inspect their structures more often; or the structure is on a particularly critical route. Elements inspected very frequently often suggest that there is an inherent aggressive defect with the structure that needs to be monitored extensively. Elements that have been subjected to 4-6 inspections (1.49%) are not typical of the population.

6. Results Analysis

6.1. Results Analysis Methodology

The approach used to analyse the results is a graphical one. The MTTFs from each condition state to the Repair and Replacement thresholds are used to evaluate if there is any evidence that the attributes should be compared separately. The process of deciding whether the evidence is significant or not is subjective and relies on the MTTF graphs. To aid in the decision making process, some of the MTTF graphs have been re-ordered because the graph axes are linear, however they represent movement through a 2-D system. The deterioration profiles are then calculated and run through simulations to aggregate the final results.

6.2. Local Environmental Factors Results
6.2.1. Structure Type

Railway bridge assets are designed in a number for configurations depending on the topology and situation. The vast majority of the assets fall into the structure types "Overline bridge" and "Underline bridge". Overline bridges are configured for the roadway to go over the structure and the railway to go beneath the structure; in this configuration the structural loading will be dominated by the roadway. Underline bridges are configured for the

railway to go over the structure and the roadway to go beneath the structure; in this configuration the structural loading will be dominated by the railway. Considering this study is focused on the asset management of railway bridges, underline bridges are of more priority as they are the responsibility of the railway management organisation.

Underline and overline bridges are subjected to different structural loading due to their configurations. Therefore an analysis was performed to ascertain if there is a difference in the way that underline and overline bridges deteriorate. There are other structure types e.g. "side of line bridge", however the vast majority of bridges (94.4%) are either underline or overline bridges. Figure 3 shows the results of the network MTTF values. The graph shows the difference in MTTF values between overline and underline bridges to the Major repair threshold. This graph shows the time bridges starting in a range of conditions take on average to reach the Major Repair threshold. The graph is used to identify if there is enough evidence to consider the attributes as independent or not. The x-axis has been re-ordered to aid in this decision making process. In general, however, the overline bridges seem to have higher MTTF values, suggesting they take more time to get from one state to another and therefore slower deterioration. The likelihood is that underline bridges will be subjected to higher loading considering they take the railway over a roadway. Therefore the structural loading will be higher, leading to more cracking, for instance, and subsequently higher rates of deterioration. The results of the analysis suggest that underline and overline bridges deteriorate differently and should be considered separately henceforth.
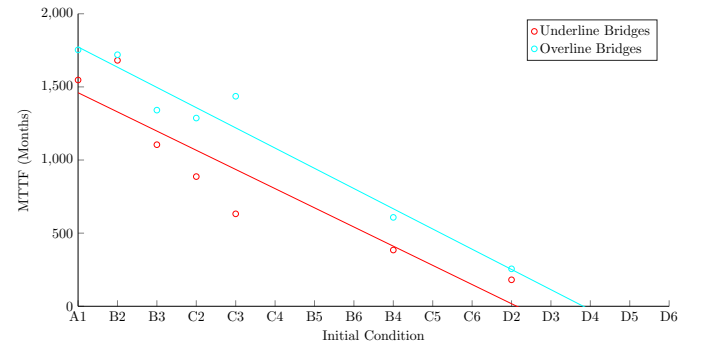

Figure 3: Difference in MTTF times of Underline bridges and Overline bridges to get to the Major Repair threshold.

6.2.2. Traffic Loading

Once a difference in structure type was determined, an analysis was carried out to see if structural loading of underline bridges was important as a deterioration factor. The premise was that the more loading the bridge was subjected to, the more severe the defects would be and therefore the faster the deterioration. Each underline bridge was cross-referenced against track location. Track data was obtained which details the total amount of Equivalent Million Gross Tonnes Per Annum (EMGTPA) which

passes over the line. This is calculated by the total amount of tonnage going over the track where each train is multiplied by factors depending on its wear characteristics. A number of influences impact the load and wear factors including: the percentage of freight on the train, the power of the tractive unit and the maximum speed of the train (Network Rail, 2011).

The results of the analysis can be seen in Figure 4. The bridges have been grouped into those which experience less than 3.5 EMGTPA, those which are subjected to between 3.5 EMGTPA and 12 EMGTPA and finally, those which experience the most severe loading at over 12 EMGTPA. The data points and trend lines are grouped by colour to indicate the MTTF values for the same set of data to two different thresholds: Major Repair and Replacement. The data points and trend lines grouped towards the bottom are to show the MTTF values to Major Repair. The data points and trend lines grouped towards the top are to show the MTTF values to Replacement.

The MTTF values presented in Figure 4 seem to suggest that there is no discernible different in deterioration rates depending on the EMGTPA. Upon reflection, the study is focusing on concrete structures which are relatively recent in bridge construction and therefore would have been built to modern standards (British Standards/Eurocode). In the Eurocode legislation for construction, each element is designed with consistent safety factors (Eurocode, 1996) and therefore the capacity of a concrete bridge is such that the EMGTPA is within specification. This analysis is included in this study because it may be that other material types do show a notable difference when looking at EMGTPA even though it was not applicable for concrete elements.
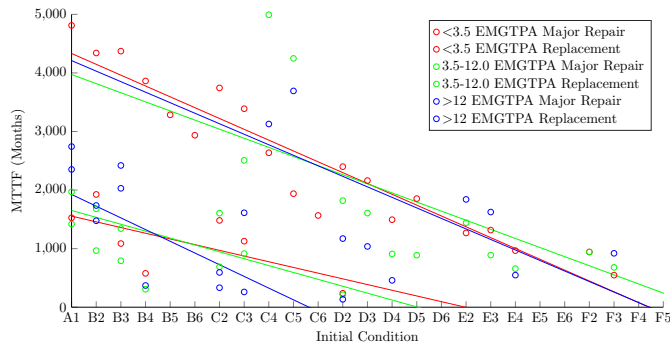

Figure 4: Difference in MTTF times of Underline bridges under different loading conditions to get to the Major Repair and Replacement thresholds.

6.2.3. Maximum Line Speed

Another LEF that is complementary to traffic loading is train speed. The proposition for this analysis was that faster trains cause larger dynamic amplification factors. The way the analysis was carried out was that the underline bridge locations were cross-referenced with track data to determine the maximum line speed of the track. Each

section of track is categorised with a maximum line speed to the nearest five Miles Per Hour (MPH). Groupings of these line speeds were performed taking into account the policy documents (Network Rail, 2014) and the line speed database.

The results of the analysis can be seen in Figure 5. The track has been grouped from those which experience very slow traffic (0-40 MPH); those which experience a low-medium traffic speed (40-75 MPH); a medium-high traffic speed (75-100 MPH) and those which are on high speed lines (100-130 MPH). Again, the data has been grouped by colour to show the same data set with the Major Repair and Replacement thresholds.

The results of Figure 5 suggests that there is no correlation between bridge deterioration and line speed. From discussion with NR experts, it seems that different track speeds are categorised as different route criticalities meaning they are subjected to different inspection and maintenance procedures as well as design requirements. This may mean that faster lines are inspected more often and therefore bridge defects may be picked up in the process.
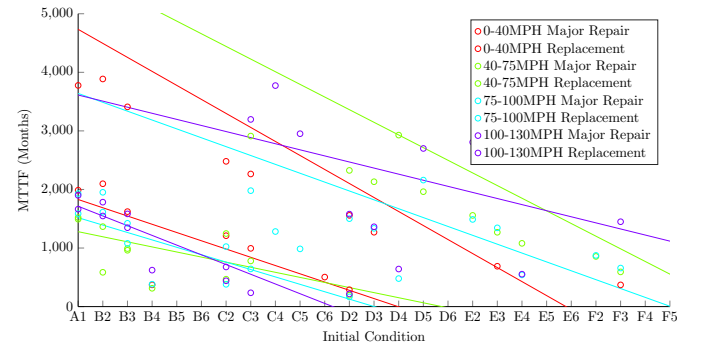

Figure 5: Difference in MTTF times of Underline bridges with different track speeds to get to the Major Repair threshold.

6.2.4. Galvanic Response

The main mechanism of concrete structure deterioration is spalling, caused by the corrosion of the reinforcement bars that make up reinforced concrete. As the reinforcement bars corrode, they expand, pushing out the cover layer of concrete, further exposing the reinforcement bars. It follows that environmental conditions that cause accelerated corrosion in metals should also affect reinforced concrete due to its reliance on the reinforcement bars. Data is available to show the galvanic response in different areas of the UK (UK Galvanizers Association, UK Galvanizers and High Chemicals UK Ltd., 2002). In each 10km^2 across the UK a corrosion rate was calculated for zinc, a popular galvanising material. These corrosion rates vary from 0.5-2.5 μm/year. Reinforcement bars in reinforced concrete are not made from zinc, but from steel and so corrosion ranks were used instead, ranging from 1-5 i.e. corrosion rate 0.5μm/year became corrosion rank 1; corrosion rate 2.5μm/year became corrosion rank 5. Figure

6 shows the map of the UK overlaid with the galvanic response information.

In practice, the bridge locations were cross referenced with the galvanic response map and then grouped according to their corrosion rank. There were no bridges to be found in corrosion rank 1 and so it was disregarded. Corrosion rank 2 only contained a minimal amount of bridges and so it was pooled together with corrosion rank 3, a much larger group of bridges. Once the bridges were grouped, each of the deterioration profiles could be calibrated and the MTTF values calculated. They were then processed into the network approach discussed in Section 3.3 and the differences in the MTTF to Major repair and replacement thresholds compared.

The results of the analysis can be seen in Figure 7. Again, the x-axis has been re-ordered to aid in the decision making process. Looking at the trend lines, it is quite evident that the corrosion levels should be considered separately. Corrosion rank 2/3 seems to have quite high MTTF values with a strong slope on the trend line. This suggests that bridges in areas of corrosion rank 2/3 seem to have higher MTTF values, relating to longer times between states and therefore slower deterioration. This would follow with expectation as a lower corrosion level would intuitively relate to slower deterioration. Similarly, corrosion rank 4 seems to trend below corrosion rank 2/3 relating to lower MTTF values, faster movement between states and therefore more severe deterioration. Finally, corrosion rank 5 seems to have the most severe deterioration. All of the MTTF values are comparatively low and the slope on the trend line is minimal suggesting that the corrosive nature of the area is such that a bridge in any condition will suffer greatly from the environment. This is in line with what is expected as corrosion rank 5 is only in places with extreme fluctuations in temperature, humidity and salinity (see Figure 6). Overall the results of the galvanic response analysis suggest that there are significant differences in bridge deterioration relating to the area of galvanic response they are situated in. Henceforth they will be treated separately.

6.2.5. Coastal Proximity

To analyse coastal proximity each of the structures with coordinates needed to be cross-referenced to a coastline map. A Global Self-consistent, Hierarchical, High-resolution Shorelines (GSHHS) database was used to plot the coastline and the coordinates of each bridge used to calculate the proximity of the bridge to the coast (National Oceanic and Atmospheric Administration (NOAA), 2015). The bridges proximity was then aligned with its corresponding element inspections. This allowed the inspection data to be split at various distances and the coastal bridges compared with the inland bridges. The distance was varied from 0.5km to 30km to ascertain the difference in MTTF values between the coastal and inland bridges.

Figure 8 shows the results of the analysis with the MTTF values. These are the mean expected times for a
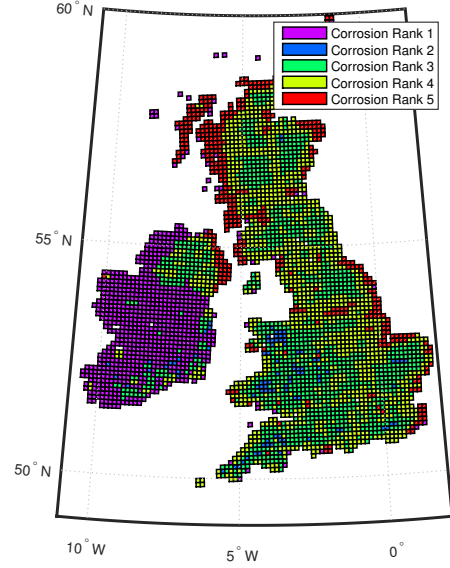

Figure 6: Different corrosion ranks in the UK (UK Galvanizers Association, UK Galvanizers and High Chemicals UK Ltd., 2002)
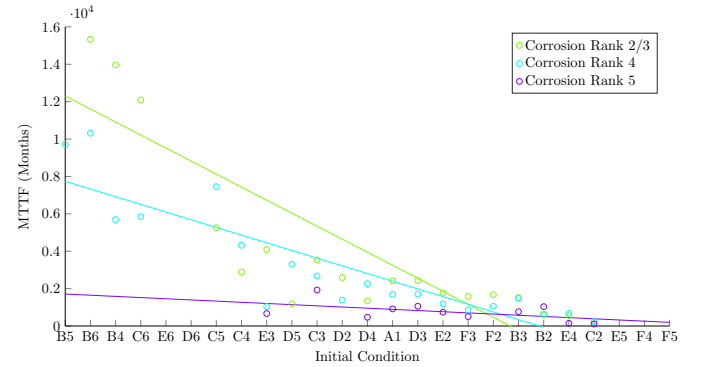

Figure 7: Differences in MTTF times for bridges in different areas of galvanic response to the replacement threshold.

bridge to deteriorate from a new (A1) condition to a state requiring Minor Intervention (see Table 2). The graph has been normalised with the inland MTTF values set at 1. What is evident is that the coastal bridges, at every position, have lower MTTF values suggesting that they move more quickly between condition states and therefore deteriorate faster. For the Major Intervention threshold, it can be seen that coastal bridges on average deteriorate roughly 35% faster than inland bridges. Additionally, it seems that bridges closer than 5km to the coast deteriorate even faster. This may be because they are directly exposed to saline humidity accelerating their deterioration.

Figure 9 shows the results of the analysis relative to the Replacement threshold (see Table 2). Again, the graph has been normalised so that the inland bridges have MTTF values set at 1. What is clear is that the coastal MTTF values are always lower than inland bridges, with MTTF values being around 30% lower for coastal bridges. The bridges closer than 5km do not seem to be as affected as in the Figure 8. This may be because the saline atmo-

sphere accelerates chloride penetration into concrete which is the precursor to spalling. Once spalling has begun, the surface concrete often falls off which directly exposes the reinforcement to corrosion. Therefore at less severe condition states, where cracking and spalling are just developing, the coastal environment is more significant. Additionally, it seems that beyond 25km the MTTF values seem to consolidate between coastal and inland bridges suggesting that the effect of the coastal environment is coming to an end.

Overall, it can be seen that there is a distinct difference between coastal and inland bridge deterioration. This means that they should be treated as distinct groups. The most intuitive way to group the data is those bridges which are adjacent to the coast and the most affected (<5km), the bridges which are moderately affected by the coastal environment (5-25km) and the inland bridges which are beyond the effect of the coastal environment (>25km).
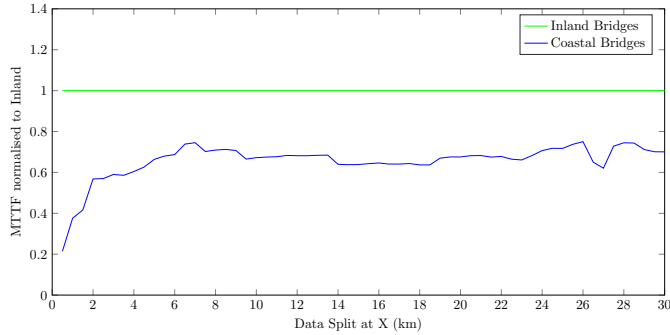

Figure 8: Relative MTTF of coastal and inland bridges from a new (A1) condition to the Major Intervention threshold.
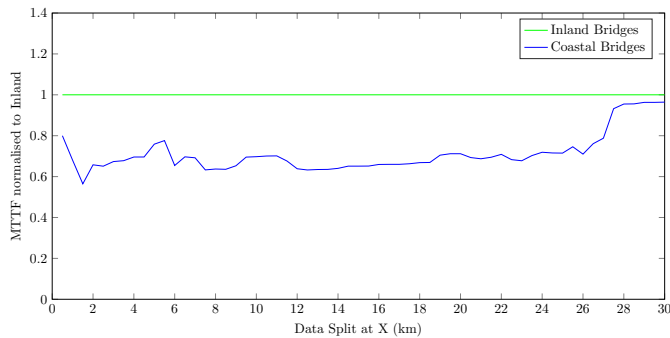

Figure 9: Relative MTTF of coastal and inland bridges from a new (A1) condition to the Replacement threshold.

6.2.6. Summary

A number of different LEFs have been analysed. Starting with the structure type, it was clear to see that overline bridges and underline bridges should be analysed separately. This decision was backed up by bridge experts who advised that they should be considered separately due to differences in possession and maintenance costs too. Underbridges, where the railway is carried over the roadway,

was seen as the most critical structure type. So analysis was done to see whether traffic loading on underline bridges played a significant role in the bridges deterioration. The analysis showed that there is no significant difference. A complementary analysis was undertaken into the track speed of underline bridges. Again, this provided no significant differences in bridge deterioration between the line speeds of underline bridges. The next factor to be analysed was galvanic response. A comprehensive map of the galvanic response rate in different areas of the UK was used and compared against bridge deterioration. The results gave a positive result showing there is indeed a difference in bridge deterioration when grouped by the different corrosion ranks. The last factor to be investigated was the effect of coastal environments on bridge deterioration. The results showed that there is a significant effect on bridge deterioration, bridges which are very close to the coast (<5km) are the most severely affected, and even those up to 25km are affected. There was some discussion to see whether the galvanic response study was correlated to the coastal proximity study. When looking at Figure 6 it is clear that the corrosion ranks are not simply a correlation of coastal proximity. Bridges that are within 5km of the coast can be in corrosion rank 2, 3, 4 or 5. Galvanic corrosion can be affected by many more factors than salinity (e.g. temperature fluctuations, pollution levels and humidity (Varela et al., 1997; Hack, 1988)). Therefore galvanic response should be considered as a separate attribute. The factors which have been determined as critical to bridge deterioration in this study are: structure type, galvanic response and coastal proximity. Table 3 shows the total numbers of concrete bridges when grouped by the LEF scenarios.

Table 3: The number of main girder elements in each category.

Structure Type	Corrosion Rank	Proximity To Coast			
		<5Km	5-25Km	>25Km	Total
Overline Bridges	2,3	1797	2062	14503	18362
	4	3347	4786	14110	22243
	5	394	316	2420	3130
Underline Bridges	2,3	1471	3524	8977	13972
	4	2024	2962	8962	13948
	5	1190	386	1171	2747
	Total	10223	14036	50143	74402

7. Incorporating Local Environmental Factors into the Petri-Net Model

A number of different modelling techniques have been used for bridge modelling (Morcous et al., 2010; Sobanjo, 1997; Arditi and Tokdemir, 1999). To be able to incorporate LEFs, a flexible modelling technique was required. The deterioration profiles were required to be dynamic so that a variety of different bridge elements could be pro-

cessed with all different situational characteristics. Andrews (2012) developed a model using Petri-Nets (PNs) which allows for the required flexibility. The development of PNs began with Petri (1962) and they have become increasingly popular. British Standards Institution (2012) explains their merits with particular reference to economic models, infrastructure models and production line models.

A PN consists of two types of nodes: places and transitions. Tokens occupy places and the state is given by their presence or absence i.e. a token occupying a place marked "as new" would indicate that a bridge is in as "as new" condition. Transitions move tokens from place to place, representing deterioration between condition states, for instance. This can be seen in Figure 10; two bridge elements are in an "as new" condition, transition t_1 replicates deterioration by moving the bridge element(s) to the good condition. Transitions can have a number of different criteria for triggering the movement of a token. For instance, with deterioration, a stochastic transition can be used which would be embedded with parameters for generating transition times. The parameters embedded in the transitions are important because they dictate the profile of the deterioration, for instance. Calibration of the transitions to historical data enables the model to mimic the real-world system more precisely.
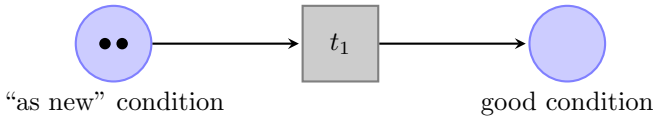

Figure 10: Simple example of a PN for a bridge with two elements.

7.1. Petri-Net Model

A PN model has been developed which uses a number of different modules. Each of the modules uses a different data calibration technique and different sources of data. The modules interact with one another to model element deterioration, inspection and maintenance. A general overview of the modules can be seen in Figure 11. The overall model is designed to aid bridge portfolio managers so that predictions are available for the types of maintenance required, the cost of the maintenance and when the maintenance will be required.

To be able to model such a complex system, a Coloured Petri-Net (CPN) approach has been used which allows the PN to be enhanced with advanced functions (Jensen, 1997). Tuple information has been incorporated into the CPN which allows tokens to be embedded with information which is critical during simulation. For example, a bridge element, represented by a token, could contain information relating to the associated corrosion rank. Then an advanced CPN transition can modify its decision accordingly i.e. change the deterioration parameters according to the type of element that is being processed. The CPN model developed is referred to as a PN model for

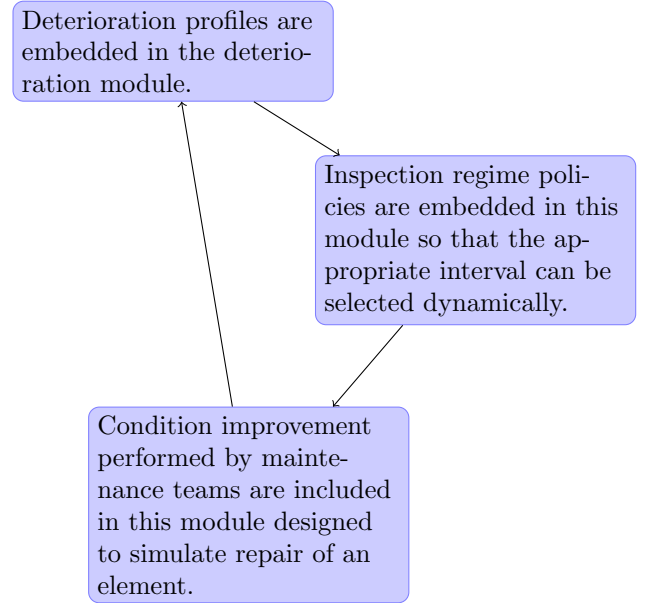simplicity. Further details about the PN model can be found in Yianni et al. (2016).

Figure 11: General overview of the PN model and its component modules. Each of the modules performs a different function and interacts with the other modules.

7.1.1. Deterioration Module

One of the most important modules of the PN model is the deterioration module. There are other modules, however this is the module the LEFs will be incorporated into. The deterioration module was initially fit with all the inspection data, this is used as the baseline or control scenario. Each of the inspections was amalgamated and then processed into MTTF values. These MTTF values are embedded into the PN transitions and used with a Monte Carlo simulation to generate transition times to move from one place to another. Each place represents one condition state of the SevEx condition scale. The simplified version of the deterioration module can be seen in Figure 12. It can be seen that there are two tokens in Figure 12, one token in condition B2 and one in condition B3. These represent two different elements on the same structure at different levels of deterioration. They are analysed separately and move around the deterioration module independently, as per CPN rules.

To incorporate the LEFs into PN model a similar approach to fitting the baseline scenario was used. The bridges that satisfied the condition (i.e. were underline bridges or 5-25km to the coast) were selected and their corresponding element inspections were used to generate the MTTF values. Where there was not enough data to calculate MTTF values for a specific scenario the baseline MTTF values were used. In the vast majority of cases there was sufficient data to provide the MTTF values required. For each scenario a database of MTTF values was

calculated which enables the model to dynamically allocate the PN transition parameters depending on the token tuple information. I.e. an element of a bridge <5km from the coast will cause the deterioration module transitions to modify their parameters accordingly so that the deterioration profile is matched.
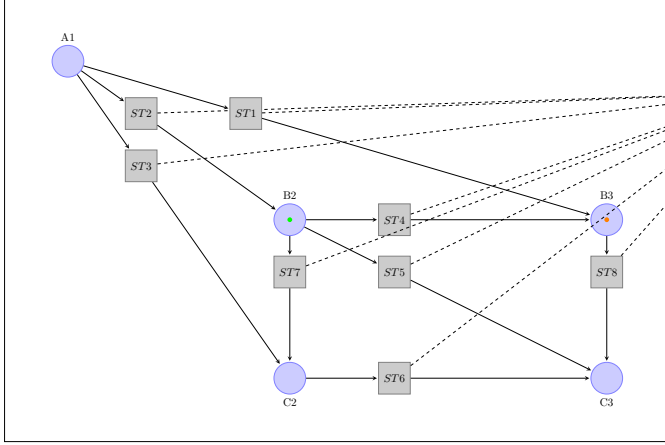

Figure 12: The deterioration module is presented in a reduced form for illustrative purposes. In full there are 31 condition states from A1, the new condition, to G6, the poorest condition.

Each of the LEFs can be run as separate simulations to be able to get results of the behaviour of each scenario. The results of each of the scenario simulations can be found in Table 4. The results detail the probability of different maintenance requirements over the 100 year simulation period. The elements begin the simulation in condition state A1 (new condition). The baseline scenario (ID 1) contains all the available data and it can be seen that on average, over the 100 year simulation, 57% of the time the element was in a condition that would only require Minor Repair.

When comparing structure types (ID 2,3) it can be seen that overline bridges seem to fare better as they spend 67% of the time in a condition state only requiring Minor Repair. Underline bridges perform similarly to the baseline condition with 58% of the time in a condition state only requiring Minor Repair. Note that ID 1 contains all the available data, however not all of this data is in ID 2 and 3 because not all bridges are underline or overline bridges, hence the disparity in the probabilities. However, all the bridges in ID 4, 5 and 6 are in ID 1; and similarly all the bridges in ID 7, 8 and 9 are in ID 1.

When looking at coastal proximity (ID 4,5,6) the bridges closest to the sea seem to degrade the fastest, spending only 48% of the time in a state only requiring Minor Repair. Those which are 5-25Km seem to fare slightly better, but those which are inland (>25km) seem to fare the best with 71%, higher than the baseline scenario.

Finally, galvanic corrosion scenarios (ID 7,8,9) are most interesting as they have the most variation. Those in low corrosion areas spend almost 73% of their time in a state only requiring Minor Repair, even better than inland bridges. The bridges that are in corrosion rank 4 spend

67% of their time in a state only requiring Minor Repair. Finally, bridges that are in the most corrosive areas (ID 9) spend only 40% of their time in a state only requiring Minor Repair, which is even lower than those <5km from the coast. These results suggest that structural deterioration is heavily dependant upon the LEFs.

8. Model Outputs including Local Environmental Factors

When simulating the PN model, each of the LEFs scenarios can be compared. The most sensible approach to comparing the scenarios is to use a like-for-like approach so that the outputs can be compared to one another. The scenarios were simulated in the PN model with a "Managed" maintenance strategy. This means that the elements were only repaired when they required Major repair or Replacement; Minor repairs were not considered. Using historical data and expert judgement, the costs are as follows: inspections cost £3,000; Major Repairs cost £40,000 and Replacements of elements costs £150,000. The elements begin the simulation in condition state A1, like new condition. Additionally, in the NR policies, inspections of structures depends on the condition of the structure. The better condition a structure is in, the less frequently the inspections are scheduled. For structures in poorer condition, they need to be monitored more closely and so the inspection frequency is closer together. However, for the scenario comparison, the inspections were fixed at every 3 years so that there is no bias. The simulation was run with a single concrete girder element, the exemplar element in this study. The results of the scenario simulations can be seen in Table 5.

ID 1 is the baseline scenario and it contains all of the available data. It can be seen that the mean cumulative cost of maintenance over the 100 year simulation period was £79,000. Overline bridges (ID 2), where the roadway travels over the railway, experiences fewer maintenance actions and therefore a lower mean cumulative cost of £36,000. In contrast, underline bridges (ID 3), seem to have many more maintenance actions and the mean cumulative cost is almost twice as much as if the element were on an overline bridge. These results show there is a significant difference in how underline and overline bridges should be managed.

When looking at the effects of coastal proximity (ID 4,5,6), it can be seen that elements that reside on bridges closer than 5km to the coast experience faster deterioration with mean cumulative costs of £151,000. This is almost double what the baseline scenario costs which shows that coastal proximity should play a pivotal role in the management of railway structures. Similarly, elements on bridges between 5-25km from the coast do not fare much better with mean cumulative costs of £137,000. Lastly, elements on bridges that are far from the coast (>25km) seem to deteriorate the slowest, requiring the least amount

Table 4: Results of the deterioration module simulation for each of the scenarios.

ID	Structure Type	Coastal Proximity (km)	Corrosion	Probability						
				Average	At 50 years			At 100 years		
				State Requiring Minor Repair	State Requiring Minor Repair	State Requiring Major Repair	State Requiring Replacement	State Requiring Minor Repair	State Requiring Major Repair	State Requiring Replacement
1	All	0 - Inf	2,3,4,5	0.5714	0.5561	0.4367	0.0071	0.0109	0.8741	0.1149
2	Overline Bridges	0 - Inf	2,3,4,5	0.6708	0.6947	0.3020	0.0033	0.0156	0.9215	0.0629
3	Underline Bridges	0 - Inf	2,3,4,5	0.5823	0.5791	0.4119	0.0090	0.0095	0.8676	0.1229
4	All	<5	2,3,4,5	0.4830	0.4428	0.5482	0.0089	0.0064	0.8888	0.1048
5	All	5-25	2,3,4,5	0.4944	0.4507	0.5306	0.0187	0.0121	0.8004	0.1875
6	All	>25	2,3,4,5	0.7121	0.7371	0.2572	0.0057	0.0339	0.8598	0.1063
7	All	0 - Inf	2,3	0.7251	0.7461	0.2472	0.0066	0.0202	0.8878	0.0921
8	All	0 - Inf	4	0.6750	0.6947	0.2967	0.0086	0.0340	0.8564	0.1096
9	All	0 - Inf	5	0.4038	0.2949	0.6449	0.0602	0.0025	0.6549	0.3425

of maintenance as their mean cumulative costs are only £28,000.

Finally, the effects of galvanic response (ID 7,8,9) seem to be quite influential. Elements on bridges in areas of low corrosion seem to deteriorate slowly, requiring little maintenance as their mean cumulative cost only amounts to £18,000, only a quarter of the baseline scenario. In stark contrast, however, elements on bridges in an area of high galvanic response (ID 9) seem to deteriorate very rapidly, requiring the most maintenance of all the tested scenarios. The mean cumulative cost amounts to £275,000 which is more then triple the cost of the baseline scenario. This shows a significant difference in maintenance costs over the simulation period. The effects of the LEFs have been quantified and there is evidence to suggest that they are significantly different that they should require different maintenance strategies.

Table 5: Cumulative cost results of scenario simulation.

ID	Structure Type	Coastal Proximity (km)	Corrosion Rank	Cumulative Cost (nearest £1000)	Relative Cost to ID 1
1	All	0 - Inf	2,3,4,5	79,000	1.000
2	Overline Bridges	0 - Inf	2,3,4,5	36,000	0.460
3	Underline Bridges	0 - Inf	2,3,4,5	71,000	0.906
4	All	<5	2,3,4,5	151,000	1.918
5	All	5-25	2,3,4,5	137,000	1.736
6	All	>25	2,3,4,5	28,000	0.355
7	All	0 - Inf	2,3	18,000	0.235
8	All	0 - Inf	4	40,000	0.512
9	All	0 - Inf	5	275,000	3.501

8.1. Model Outputs

Simulation can be performed with the standard NR maintenance policies and the LEFs. Some exemplar model outputs can be seen in Figure 13 and Figure 14. The examples in Figure 13 show the elements all starting as new

(A1) and then deteriorating at different rates. The saw-tooth pattern is evident because of the inspection interval which then may lead to intervention. The inspection interval is 12 years for elements in good condition, however towards the end on the simulation the elements drop to a moderate condition and so the inspection interval drops to 6 years. Had they fallen to a poorer condition, the interval would have changed to 3 year intervals.

Similarly, nominal and cumulative cost outputs (seen in Figure 14) show when maintenance is required, the type of maintenance and the typical cost, calculated from historical data. Again, it can be seen that the costs arrive in waves, originating from the inspection frequency - maintenance can only be scheduled when the condition of the elements is known. In some circumstances, as evident in figure 14b, there are costs that are out of sync with the others; these originate due to rapid deterioration of the element. When the element deteriorates to a poorer condition state during the intervention scheduling delay; the maintenance teams attempt to carry out the intervention, however the defect has worsened beyond what they were expecting and so they will not have the available time/resources to carry out the repair and so it must be rescheduled. Elements in situations with more severe LEFs will be subjected to this more often as is evident when comparing figures 14a, 14b and 14c. The result of an element being in a more severe LEF is that it will require more maintenance, increasing its costs, as well as being harder to manage due to the scheduling delay issue. For railway portfolio managers this situation can introduce further complexity when attempting to maximise maintenance team effectiveness.

9. Conclusion

There are many studies that focus on bridge deterioration modelling (Morcous et al., 2002; Sobanjo, 1997; Arditi and Tokdemir, 1999) with a brief mention of the "other" factors that were not considered. Many authors suspect
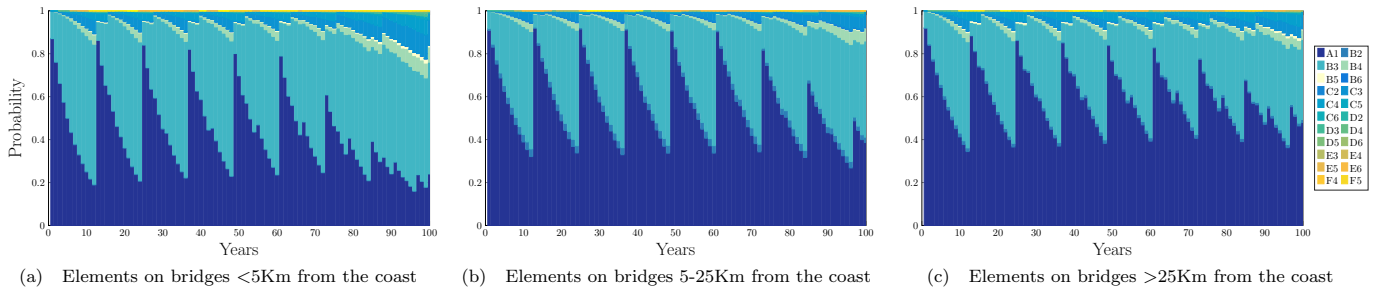
(a) Elements on bridges <5Km from the coast (b) Elements on bridges 5-25Km from the coast (c) Elements on bridges >25Km from the coast

Figure 13: Graphs to compare the condition over time of various scenario model outputs.

(a) Elements on bridges in galvanic response areas rank 2,3 (b) Elements on bridges in galvanic response areas rank 4 (c) Elements on bridges in galvanic response areas rank 5
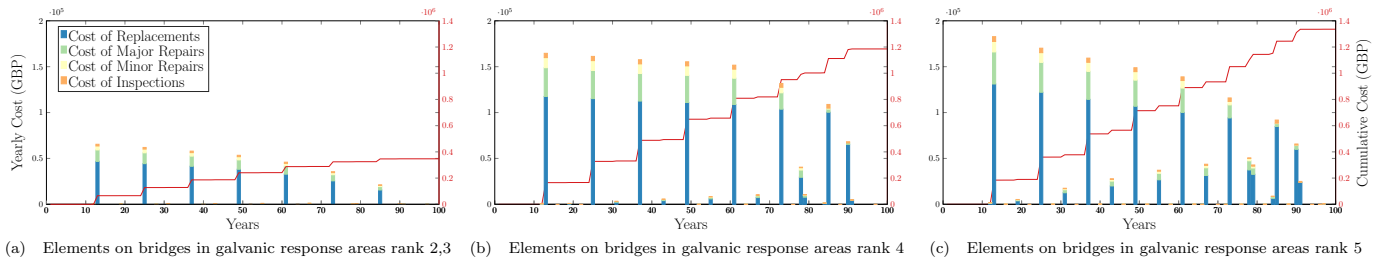
Figure 14: Graphs to compare the cumulative cost of various scenario model outputs.

that these other factors may play key roles in the method of the deterioration, sculpting the deterioration profile in a different way. However, to conduct a study into these other factors requires a large amount of data, a suitable methodology and a modelling framework that is flexible enough to accommodate them.

A number of different factors were investigated, using all the available data: from historical bridge data to corrosion data and even track line data. A number of key factors were identified, denoted as Local Environmental Factors (LEFs). The deterioration profile of each LEF scenario could then be calculated. This enabled it to be directly embedded into the PN model. With this capability it allows dynamic simulation of elements with different characteristics to be analysed. Each of the LEF scenarios was then run in a comparative simulation which produced results from which the most important factors could be identified.

In this study it was identified that the structure type, coastal proximity and the galvanic response rank were the key factors. There were key findings for bridge managers including the effect of the coastal environment. It was found that bridges closer than 25km to the coast deteriorate at a rate 1/3rd higher than inland bridges. Understanding the impact of the coast and its extent inland is vital information for bridge portfolio managers.

Another development which is integral to the management of railway bridges is the impact of galvanic response which seemed to produce the most dramatic change in cumulative cost over the simulation period. Elements in areas of low galvanic response seem to deteriorate much more slowly accruing only a quarter of the baseline cost

over their lifetimes. However, elements in areas of high galvanic response seem to suffer from accelerated corrosion and incur more then three times the cost of the baseline scenario. For infrastructure operators and managers, this result could have a large impact.

In conclusion, a novel approach to comparing two 2-D condition state systems was developed. From which a number of factors affecting bridge deterioration were investigated. From those, the key factors were identified and each of the deterioration profiles calculated. This allowed the PN to be enhanced with the LEFs which enables the ability to dynamically change the profile of the element deterioration based on the particular element characteristics. Finally, an investigation with a comparative simulation was carried out which identified which of the LEFs would be the most financially important to bridge portfolio managers.

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