Computers and Concrete, Vol. 2, No. 6 (2005) 439-454
DOI: http://dx.doi.org/10.12989/cac.2005.2.6.439

Concrete bridge deck deterioration model using belief networks

Hrodny Njardardottir† and Brenda McCabe‡

Department of Civil Engineering, University of Toronto, Toronto, Canada

Michael D. A. Thomas‡†

Civil Engineering, University of New Brunswick, Fredericton, Canada

Abstract. When deterioration of concrete is observed in a structure, it is highly desirable to determine the cause of such deterioration. Only by understanding the cause can an appropriate repair strategy be implemented to address both the cause and the symptom. In colder climates, bridge deck deterioration is often caused by chlorides from de-icing salts, which penetrate the concrete and depassivate the embedded reinforcement, causing corrosion. Bridge decks can also suffer from other deterioration mechanisms, such as alkali-silica reaction, freeze-thaw, and shrinkage. There is a need for a comprehensive and integrative system to help with the inspection and evaluation of concrete bridge deck deterioration before decisions are made on the best way to repair it. The purpose of this research was to develop a model to help with the diagnosis of concrete bridge deck deterioration that integrates the symptoms observed during an inspection, various deterioration mechanisms, and the probability of their occurrence given the available data. The model displays the diagnosis result as the probability that one of four deterioration mechanisms, namely shrinkage, corrosion of reinforcement, freeze-thaw and alkali-silica reaction, is at fault. Sensitivity analysis was performed to determine which probabilities in the model require refinement. Two case studies are included in this investigation.

Keywords: concrete deterioration; probabilistic reasoning; ASR; diagnostic model.

1. Introduction

Deterioration of our civil infrastructure has resulted in a significant portion of municipal, provincial and federal budgets being allocated to the repair, rehabilitation and reconstruction of existing facilities. To effectively allocate resources, it is important to diagnose the cause of deterioration and decide the most cost effective and appropriate method of restoration. One such infrastructure component is concrete bridge decks. Bridge deck deterioration may be caused by chlorides from de-icing salts, alkali silica reaction (ASR), frost damage, shrinkage, and carbonation of the concrete. Deterioration can also be due to poor design and detailing, inappropriate materials, poor construction quality, inadequate maintenance or any combination thereof.

† MASc
‡ Associate Professor, Corresponding Author, E-mail: mccabeb@civ.utoronto.ca
‡† Professor

It becomes evident that diagnosis of deterioration is not always a simple task. Some of the mechanisms of deterioration are not fully understood, and are complicated if more than one mechanism is at work. Expert opinion usually plays an important role in the diagnostic process. A system to help with the inspection and evaluation of concrete bridge deck deterioration could be used to facilitate the process. Bridge maintenance systems have been developed to evaluate the condition of bridges and their need for rehabilitation within the context of budgets and standards. PONTIS is a widely-used bridge management system based on Markovian chains. Although it considers the interactions of the bridge elements, it does not consider the interaction of deterioration mechanisms (Sianipar and Adams 1997). BRIDGIT builds on PONTIS by providing greater detail. Although both systems aim to minimize life cycle costs, their approach is reverse: PONTIS is considered a top-down modeling approach starting with budgets and standards, whereas BRIDGIT is bottom up, starting with information on projects and the estimated cost to complete them (Small, *et al.* 2000). Both systems store inspection data and evaluate the cost of maintenance and rehabilitation options, however, neither of them model deterioration mechanisms.

LeBeau and Wadia-Fascetti (2000) developed a structural analysis model using a fault tree. As a graphical unidirectional depiction of the various failure paths that lead to an undesirable outcome, the fault tree model offers a systematic method of organizing the element interactions that contribute to bridge deterioration. This probabilistic model advances previous systems in that it considers the effect of one bridge element on another, however, it does not consider conditional relationships between variables nor causal mechanisms of failure. Instead, it is used for structural analysis whereas the model proposed by the authors is for diagnosis of deterioration.

A bridge maintenance system designed for the Ontario Ministry of Transportation (Thompson, *et al.* 2000) contains a knowledge-based module that captures the thought processes used by inspectors to diagnose bridge deterioration. The limitation of this system is that it is based upon a decision tree; therefore, uncertainty or missing information is not easily handled. Thompson, *et al.* 2000). state that the Markovian condition model limits accurate predictions in that it assumes the next state is dependent entirely on the previous state and average conditions, and not on external or undetected phenomenon. At the time of writing, the model was unavailable for comparison with the model presented in this paper.

Developing a model that can distinguish between several deterioration mechanisms is a challenge, but could be extremely useful. Further, if the application of the model were automated, then it would fit into future bridge management systems where quantitative parameters describing the bridge condition could be determined by sensors and collected by wireless systems.

Integrating expert knowledge with theoretical knowledge in a diagnostic tool can be difficult. A relatively new modeling environment referred to as Bayesian belief networks or belief networks has been used for this purpose.

1.1. Objective

The objective of the research described in this paper was to model concrete bridge deck deterioration mechanisms so that the knowledge could be used to diagnose the cause of the observed deterioration. The model was limited to four deterioration mechanisms: drying shrinkage, alkali-silica reaction (ASR), freeze-thaw, and steel reinforcement corrosion. Causal factors such as design details, quality of installation, and maintenance were also considered. To facilitate the use of the model by practitioners, a graphical user interface was developed that allowed the input of observed symptoms accompanied by images to demonstrate the symptoms.

This paper is organized to first introduce the modeling environment of belief networks. Next, each of the four deterioration mechanisms are briefly discussed and modeled. To test the model, two case studies are described and diagnosed using the model, and the results were compared to expert diagnosis. Throughout this paper, variables/nodes within the network will be denoted with italic.

2. Introduction to belief networks

Belief networks were first developed in the 1970s. A pivotal publication by Pearl (1988) launched an explosion of research in the area. Belief networks are probabilistic graphs that exploit the properties of Bayes' Theorem, shown in Eq. (1) where $P(A)$ denotes the probability of A=true, | denotes *given*, and $\wedge$ denotes *and*. The symmetry of Bayes' Theorem allows probabilities to be propagated forward and backward. This property presents the opportunity to develop a causal model for deterioration, and then to reverse the logic and use it as a diagnostic model.

$$P(A|B) = \frac{P(A \wedge B)}{P(B)} = \frac{P(B|A)P(A)}{P(B)} \tag{1}$$

The belief networks consist of nodes representing the variables in a domain, and of directional arcs (arrows) that link the nodes to indicate conditional dependence. Conditional dependence is quantified as the probability of a variable's state given some combination of states of its parents.

There are four major steps in developing a belief network (Poole, *et al.* 1998). A simplified belief network to determine if the concrete structure is suffering from ASR is shown in Fig. 1 for demonstration purposes. First, the variables and their states are identified. This requires the model domain to be well scoped such that the variables that define the domain are mutually exclusive and collectively exhaustive. In Fig. 1, the variables are *reaction rim, exposed to moisture, map-crack, gel in cracks, total alkali content, area of known reactive aggregate* and *ASR.* In many cases, the variables are binary with states such as true/false or yes/no. However variables can have as many states as necessary to appropriately support the variable definition. In this example, all the variables are binary with true/false states, except *total alkali content*, which has three states: <2.5 kg/m^3, 2.5-3.5 kg/m^3, and >3.5 kg/m^3.

Second, arcs are placed to represent conditional dependence between the variables. In this example, the parents of *ASR* are *exposed to moisture*, *total alkali content* and *area of known reactive aggregate.* They affect the likelihood of the presence *ASR* and the presence of *ASR* in turn results in *map-crack, reaction rim* and *gel in cracks,* called the children of *ASR*.

Third, the probabilities are evaluated: prior (unconditioned) probabilities for nodes without parents (i.e. orphans) such as *exposed to moisture*, *area of known reactive aggregate*, and *total alkali content*, and conditional probabilities for the others. *ASR* has three parents and the number of combinations of parent states is twelve, with each requiring a probability. The probabilities can come from data or from experts where data are unavailable. There is considerable debate on whether single (e.g. Morris 1986, French 1986) or multiple experts should be consulted. Multiple experts require a means of combining the opinions in a rational way (e.g. Stiber, *et al.* 2004). In this research, a single expert was used.

Fourth, the model is verified and validated. This step is common for all model developments. For more detailed discussion of the functionality of belief networks, see McCabe, *et al.* (1998) or

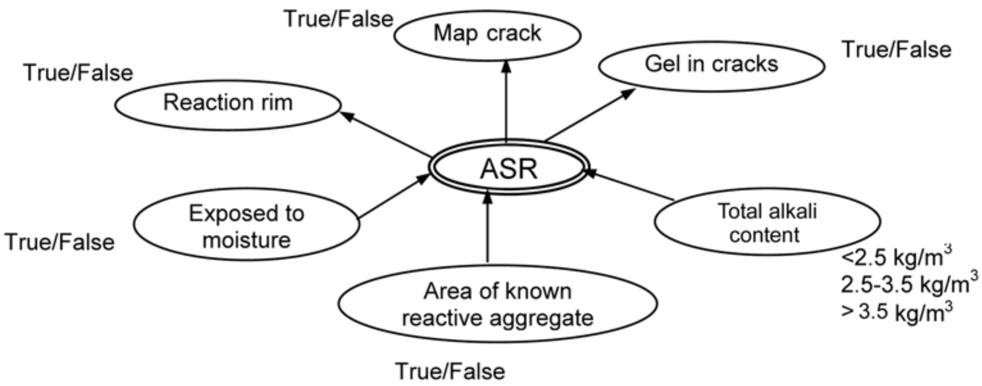

Fig. 1 Simplified ASR network

Table 1 Probabilities for ASR=True

	Total alkali content		
	< 2.5 kg/m^3	2.5- 3.5 kg/m^3	>3.5 kg/m^3
Exposed to moisture area known for reactive aggregate	10	20	30
Exposed to moisture not an area known for reactive aggregate	5	10	10
Not exposed to moisture area known for reactive aggregate	5	5	5
Not exposed to moisture not an area known for reactive aggregate	5	5	5

Jenson (1996).

Table 1 shows the expert-assigned probabilities for *ASR*. If the bridge deck is exposed to moisture, the geographic area is known for reactive aggregate, and the total alkali content of the cement is <2.5 kg/m^3, then the probability of ASR is 10%. But if the total alkali content of the cement is >3.5 kg/m^3, then the probability of ASR increases to 30%. Note that if there is no exposure to moisture, the other conditions have no effect on the probabilities due to the necessity of moisture for ASR to occur, as shown in the bottom two rows Table 1. The probability is not 0% because there is a chance that moisture is available, but is not obvious to the observer.

An advantage of belief networks is that both historical data and expert opinion can be incorporated in the model. This is important where data are not available for the development of purely numeric models. In most cases, the definition of variables, their states, and the identification of conditional dependence relationships require expert input. Where data are available, the probabilities can be extracted; otherwise expert opinion is used.

The belief network environment was used in this research for several reasons. First, the graphical format facilitates model development because of its intuitive nature. This is especially helpful to experts participating in model development who do not have comprehensive training in probability theories or in knowledge based systems. Second, changes to the model can be isolated because changes in the variables or in the relationships between variables will only affect the nodes immediately attached and not require a complete review of the model. For example, if *exposed to moisture* is removed from the model, then the only node that would be affected is *ASR* because it is the only node connected to *exposed to moisture*. Third, integration of theoretical, empirical and expert knowledge can allow the development of useful models before all aspects of a particular process are completely quantified. Finally, belief networks enable reasoning under uncertainty. The model output is the probability of each deterioration mechanism after input of the observed deterioration evidence. The user can then judge which mechanism(s) is (are) most likely causing the deterioration according to the probabilities.

3. Building the model

3.1. Variables: their states and connection

The model was developed after a thorough review of the available literature, which included text books, technical papers, and industry documents. The model consists of five integrated sections i.e., four deterioration mechanisms (corrosion, alkali-silica reaction, freeze-thaw and shrinkage) and a miscellaneous category, each of which has been isolated for discussion purposes. In the following figures, the nodes were classified as input (regular circles), output (double circles) and intermediate (dashed circles). The inputs and outputs are those directly related to the user interface. Intermediate variables do not represent diagnostic variables, but are required to complete the logic of the deterioration mechanisms.

It is important to note that the model can proceed with its evaluation with incomplete information. Specifically, the model uses *a priori* probabilities to complete its evaluation. When new data are available, they can be input to the model and the probabilities will be reassessed given the new information.

3.2. Corrosion

Fig. 2 shows the model section for reinforcement corrosion. The electrochemical degradation of steel occurs when O_2 and H_2O are present, and the passivity of the steel is destroyed by either carbonation of the concrete or by the presence of chloride ions at the steel surface (Broomfield 1997). Cracks, joint leaks, inadequate waterproofing overlay, and high *w/c* ratio are all *paths* for moisture, oxygen, CO_2 and chloride ions to travel to the rebar. Porous concrete is also susceptible to moisture and salt intrusion by diffusion. Very few bridge decks have completely watertight expansion joints, so de-icing salt is allowed to run through the deck. Waterproofing overlays can be installed on decks but it may be damaged during or after construction, making them less effective.

Covermeter testing establishes the adequacy of the concrete cover for steel protection. The concrete cover is not as important for the other deterioration mechanisms, so c*overmeter testing* is separated from the other paths, as shown in Fig. 2. The threshold for chloride is lower if the concrete is already carbonated at the cover depth. If the presence of chlorides in the surrounding environment is unlikely, or if chloride is not found in the analyses, tests for *carbonation at cover depth* should be carried out, such as *phenolphthalein* (Emmons 1993).

Because steel increases volume as it corrodes, *cracks over rebar, rust stains, delamination,* and *spalling* are all signs of corrosion. The *copper/copper sulphate half cell test* was incorporated into the model to help confirm the existence of electrochemical reaction. ASTM (1998) guidelines were used such that measured potentials less than -0.35V indicate a high probability of corrosion, whereas potentials greater than -0.2V indicate a low likelihood of corrosion.

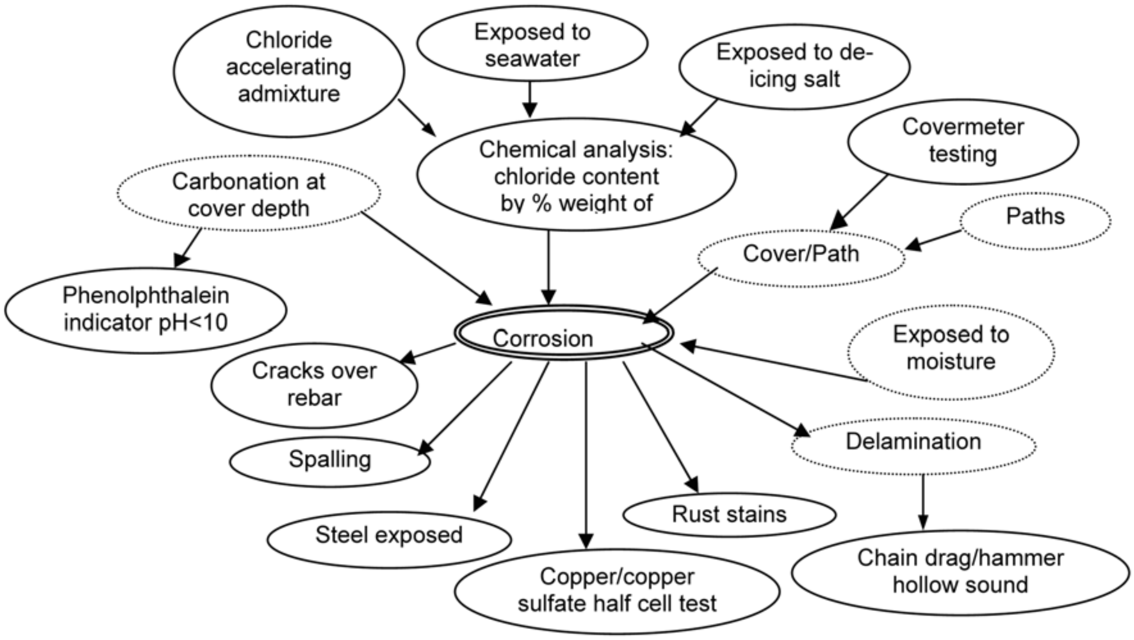

Fig. 2 Corrosion variables

3.3. Alkali-silica reaction (ASR)

As shown by the arrows coming into the ASR node in Fig. 3, ASR requires three components to occur, namely a reactive aggregate, sufficient quantities of alkalis (typically in the concrete pore solution), and sufficient moisture, perhaps through *paths*. Documentary evidence of the *total alkali content* of the cement used would be helpful information in the diagnosis process.

Alkali-silica gel has the capacity to imbibe water and when confined by the surrounding hydrated cement paste, internal pressures may eventually lead to cracking and disruption of the hydrated cement paste. Cracks due to ASR are often characterized by the *line of crack having a dark discoloration* with an adjacent zone of light colored concrete or s*light surface discoloration associated with some cracks*.

If ASR is particularly active, *colorless, jelly like exudations associated with some cracks* can be observed. Although initially colorless, this exudation (gel) carbonates on exposure to the atmosphere and is observed as *white exudations around some cracks*. *Damp patches* are usually visible at the junction of cracks. *Pop-outs* can be caused by the reaction of aggregate particles close to the surface and are easily confused with pop-outs due to frost damage. Expansion due to ASR may result in *deformations, relative movement* and *misalignment, separation of adjacent concrete members,* closure of joints causing *extrusion of jointing materials* and *spalling of concrete at joints*.

ASR gel is often invisible to the unaided eye and may therefore go undetected in structures for some time (Stark 1991). Core samples can be examined to identify symptoms of ASR, features that provide evidence of reaction, and emanation of expansive forces. As more than one test can be used to identify *gel in cracks*, *crack/microcrack* and *reaction rims*, three intermediate variables were created as child variables of *gel in cracks*, namely *petrography test: gel in cracks*, *core examination:*

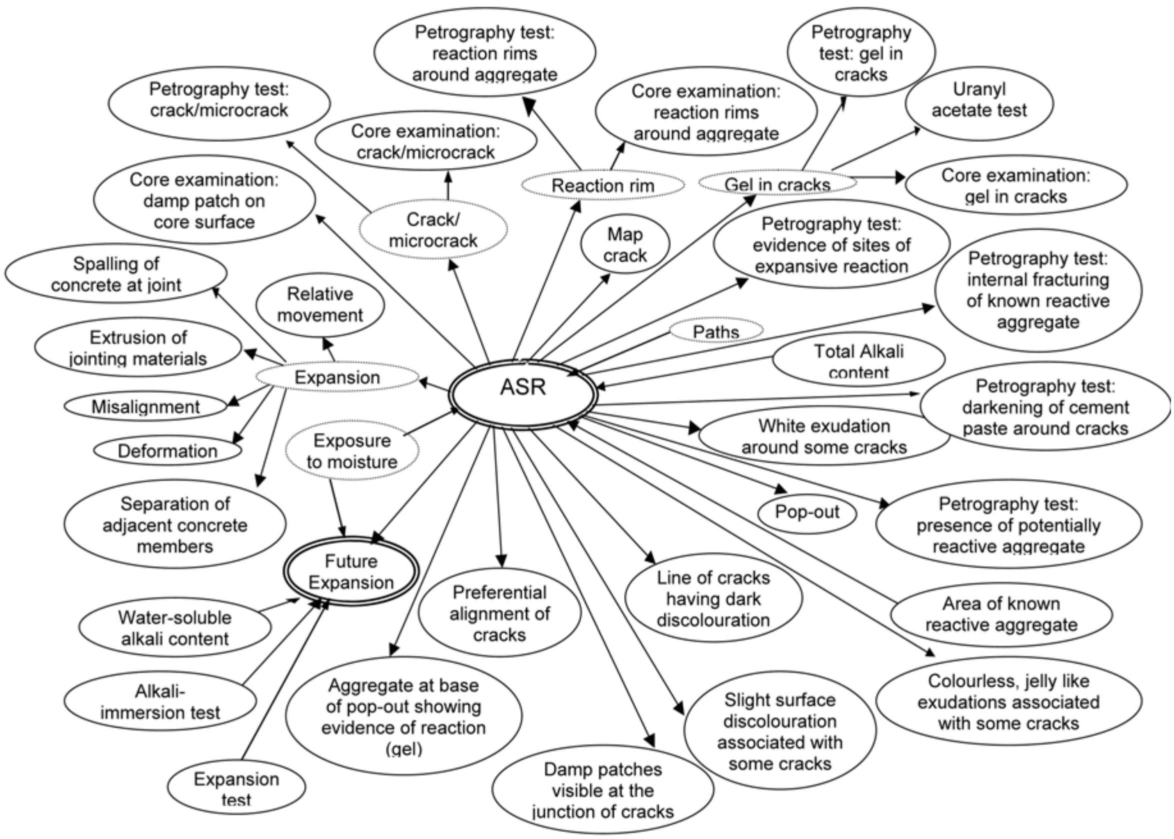

Fig. 3 Alkali-silica reaction variables

gel in cracks and *uranyl acetate test*.

The risk of *future expansion* can be assessed by three laboratory tests and is dependent upon whether ASR has already occurred. The *water soluble alkali test* indicates the remaining active alkalis in the concrete; the *expansion test* determines the potential for further expansion of concrete due to ASR; and, the *alkali immersion test* indicates the presence of reactive aggregate in concrete (Wallbank 1989).

3.4. Freeze-thaw damage

As shown in Fig. 4, two conditions are necessary for freeze-thaw damage to occur: *exposure to freezing temperature* and *exposure to moistur*e. *Paths* provide easy access for moisture and make the situation more severe. The presence of *frost susceptible aggregates* in the concrete increases the danger of freeze-thaw damage even if the concrete is air entrained (Neville 1997). Freeze-thaw damage may include *scaling, spalling* in delaminated areas, and *pop-outs* of aggregates located near a surface (Mays 1992). *D-cracking* (a series of fine cracks on the surface of the concrete adjacent and parallel to transverse joints) and *map cracking* may also be evident.

Air void analysis can determine if the air entrainment and the spacing factor are satisfactory. The spacing factor, defined as the average distance between any point the cement paste and the nearest air

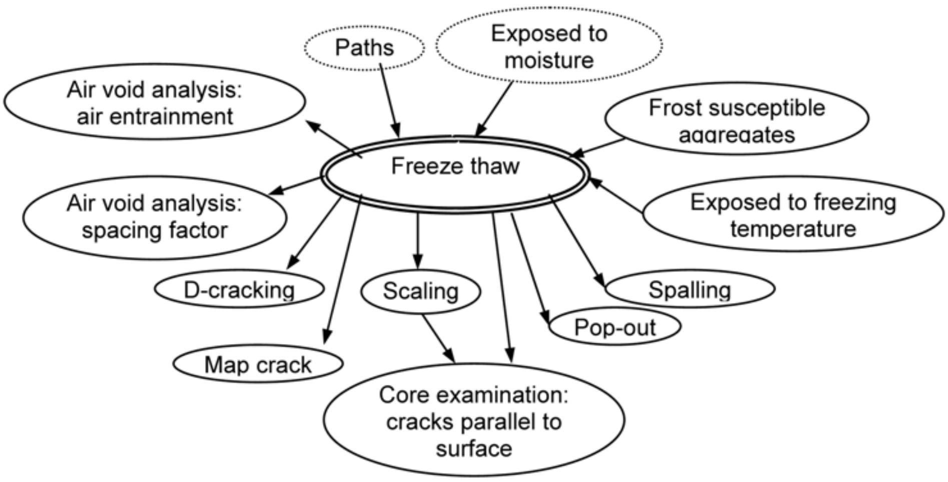

Fig. 4 Freeze thaw damage variables

bubble, should be <230 μm, according to the Canadian Standards Association CSA A23.1 document.

3.5. Shrinkage

Water in excess of that required to complete hydration is necessary to give the mix sufficient workability to enable it to be compacted easily. Some of the excess moisture is gradually released to the atmosphere, accompanied by a reduction in volume known as shrinkage. As shown in Fig. 5 *total cement content, w/c ratio* and *inefficient joint* are factors related to *shrinkage*. Shrinkage increases as the cement content increases (assuming constant w/c ratio) because a larger volume of hydrated cement paste is available to shrink (Neville 1997). *Total cement content* of the mix can be found in documentary evidence if they are available or it can be estimated from *petrography test: w/c ratio*. If joints are not provided or they are unable to accommodate for the extent of the movement, cracks will occur (Kay 1992).

If shrinkage is restrained, tensile stresses develop and cracking usually results. *Transverse cracks* develop at right angles to the long dimension of the member. *Map cracks* are fine, interconnected cracks on a concrete surface resembling a road map. *Longitudinal cracks* develop parallel to the length of the member. *Crack perpendicular to the surface* refers to cracks that extend from the surface inwards.

3.6. Miscellaneous

Paths for moisture and salt to enter the concrete include leaking bridge deck, high concrete permeability due to a high *w/c ratio,* and *cracks*, as shown in Fig. 6. *Cracks* is a collecting variable for cracks produced by shrinkage and joints filled with debris. If expansion joints become filled with debris, the deck may *crack around joints* and s*pall*. Cracks due to ASR, freeze thaw damage and corrosion are not connected to *cracks* even though they also create paths by which oxygen and moisture can gain access to the reinforcement because their connection would create a logical cycle, thereby violating the acyclic constraint of the belief network.

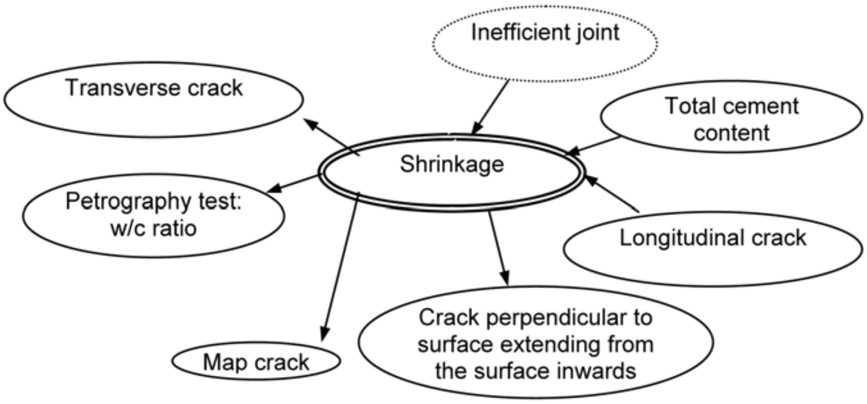

Fig. 5 Shrinkage variables

The two main objectives in sealing bridge expansion joints are to prevent the passage of water through the deck, and to prevent the intrusion of debris into the joint itself, which may render it inoperative. The most common joint defects are *deteriorated joint sealer* and *lack of sealer*. A good indication of deck joint leakage is when deck soffits and edge beams show *water runs or staining*. *Unsealed joints* can also create durability problems. In this model, only joints filled with sealant designed to prevent the flow of water and debris through the joint are considered. *Inadequate design detailing* can result in *shallow slopes*, *poor location of drains*, *unsealed joints*, and *inefficient joints*

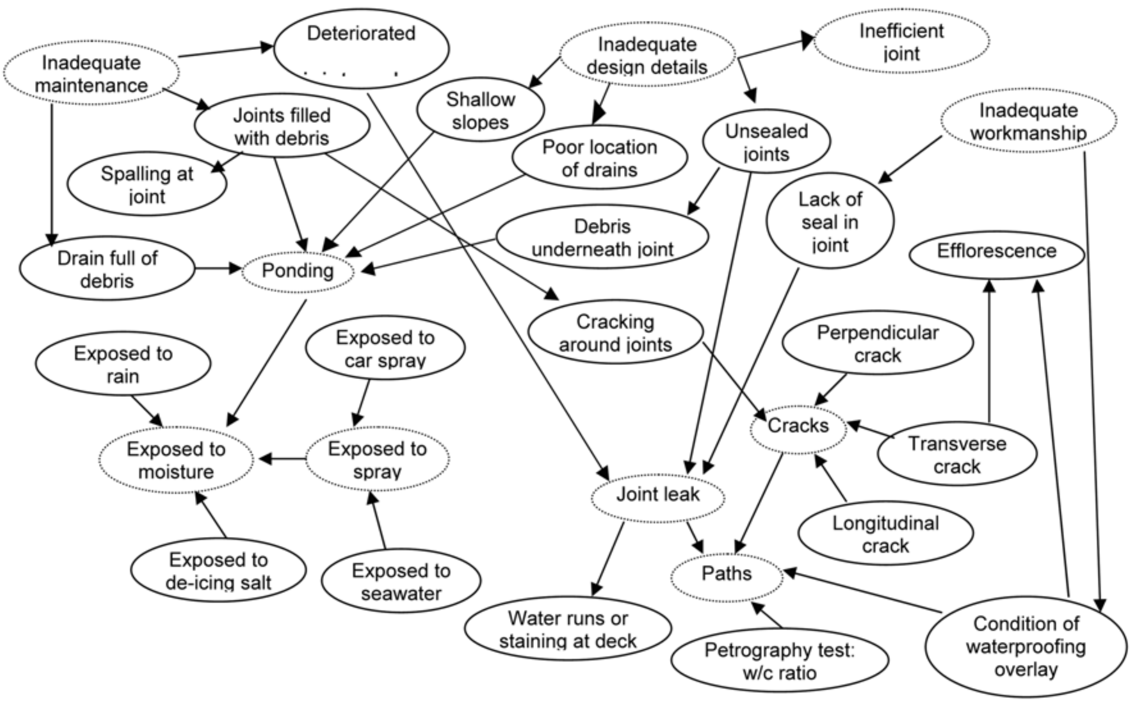

Fig. 6 Miscellaneous variables

which can cause *ponding*, *joint leakage* and contribute to shrinkage deterioration.

Although properly proportioned, placed, and cured concrete is satisfactorily wear resistant, a wearing course and *waterproofing overlay* are often used on bridge decks. The relatively porous asphalt wearing course can trap salt-laden moisture and promote deck deterioration, therefore, an asphalt surface treatment, membrane or other deck sealer is often applied prior to the asphalt. The *condition of the waterproofing overlay* can be examined by looking at sawn asphalt samples from the deck. *Efflorescence* is often observed at transverse shrinkage cracks in deck soffits and edge beams, indicating that the waterproofing is not always as effective. A high standard of construction *workmanship* is necessary to ensure that the waterproofing system is initially watertight. Other defects may include settlement caused by inadequate compaction of the subgrade, or inadequately vibrated concrete during placement resulting in internal voids and honeycombed surfaces.

Ponding can result from *poor location of drains*, *shallow slopes* and *drain full of debris*. Ponding can also result from *debris filled joints* and *debris collected under open joints*, but is not as severe as the other ponding conditions. Debris filled joints and debris that collects beneath open deck accumulate dirt and other roadway debris and become clogged (Herodotos, *et al.* 1995), which can deteriorate the adjacent deck. Ponding water promotes rapid concrete deck deterioration. Periodic *maintenanc*e such as cleaning and flushing of concrete decks, drains, and expansion joints is necessary to ensure proper deck drainage and to prevent ponding. As shown in Fig. 6, *ponding* connects these variables, and *exposed to moisture* connects all the moisture related variables. Spray from passing cars can cause chloride contamination, especially from traffic passing under the bridge, which is a major source of contamination of deck soffits (Wallbank 1989). Spray from seawater is also a source of chloride contamination.

3.7. Assignment of probabilities

With the variables and relationships established, the next step was to assign probabilities to each node. Probabilistic data are not generally available in this domain, so expert knowledge of one of the authors was used for the initial development of the model. In total, 25 prior and 276 conditional probabilities were needed. Connecting variables, such as *paths for moisture/salt, exposed to moisture*, *cracks*, *joint leak* and *ponding* were created to reduce the number of probabilities using a divorcing technique. The authors wanted to know which nodes were most sensitive to changes in the probabilities, as they would be the focus of future data collection efforts to verify the expert's estimate of the probabilities.

3.8. Sensitivity analysis

Two levels of sensitivity were investigated: instantiation and probabilities. First, the instantiated or evidenced nodes with the greatest influence on the output nodes were identified using entropy and are referred to as the influential nodes. Entropy may be used to measure the amount of information a certain piece of evidence brings with the objective of reducing the uncertainty of the hypothesis e.g. H_0: ASR is the cause of deterioration. The general equation for entropy (*Ent*) (Jensen 1996) is shown in Eq. (2), and the change in entropy given evidence (*e*), also referred to as the weight of evidence (*WOE*), is shown in Eq. (3). In the two equations, $\neg$ denotes the logical 'not', | means 'given', and *H* represents the node whereas *h* implies the state. A threshold value for the reduction in entropy was arbitrarily set at 15%; if the entropy at an output node reduced by more than 15%, then the influential node was tagged for further analysis.

$$Ent(P(h)) = \sum_{h \in H} p(h) \log_2(P(h)) \tag{2}$$

$$WOE(P(H)) = \log P(h) - \log P(\neg h) \tag{3}$$

$$WOE(P(H|e)) - WOE(P(H)) = \log \frac{P(e|h)}{P(e|\neg h)}$$

The analysis was divided into four parts: *ASR, Corrosion*, *Freeze-thaw*, and *Shrinkage*. The results for ASR are shown in Table 2. Six variables over 15% were taken to the second stage of sensitivity analysis. Note that they are all descendents of *ASR*. The other results were:

- *Corrosion* had influential variables *rust stains*, *copper half cell*, *crack over rebars* and *spalling*
- *Shrinkage* had influential variables *crack perpendicular to surface extending from the surface inwards* and *transverse crack*

In the second level of sensitivity analysis, the conditional probabilities at the influential nodes were varied within levels suggested by experts to determine their effect on the output. For example, Table 3 shows how the probabilities were altered to determine the effect of *Gel in cracks* on *ASR*. In Test 1, the *P*(*Gel in cracks*=true|*ASR*=true) was increased from 80% to 95%. This resulted in insignificant changes in the probability of the output node where the *P*(*ASR*=true|*Gel in cracks*= true) increased by 5% from 40% to 45%, and *P*(*ASR*=true|*Gel in cracks*=false) decreased by 2% from 3% to 1%. In Test 2, the *P*(*Gel in cracks*=true|*ASR*=true) was decreased from 80% to 60%, thereby reducing the strength of the relationship. Again, the change was minimal, with *P*(*ASR*=true|*Gel in cracks*=true) decreasing slightly from 40% to 33%, and *P*(*ASR*=true|*Gel in*

Table 2 Entropy change for ASR

Node	Reduction in Entropy
Gel in cracks	28.2%
Crack/microcrack	22.8%
Petrography test: evidence of sites of expansive reaction	20.5%
Petrography test: internal fracturing of known reactive aggregate	19.6%
Map-crack	15.7%
Petrography test: gel in cracks.	15.6%
Damp patches visible at the junction of cracks.	14.0%
Preferential alignment of cracks.	14.0%
Reaction rim	13.5%
Petrography test: darkening of cement paste around cracks.	13.5%
Core examination: damp patch on core surface.	13.5%
Petrographpy test: crack/microcrack.	13.1%
Slight surface discoloration associated with some cracks.	8.3%
Colorless, jelly like exudations associated with some cracks.	8.3%
White exudation around some cracks.	8.3%
Core examination: crack/microcrack.	8.0%
Petrography test: reaction rims around aggregates.	7.5%

Table 3 Second stage sensitivity analysis

	Change in Influential Node (%)		Resulting Change in Output Node (%)	
	P(Gel in cracks= true | ASR=true)	*P*(Gel in cracks= true | ASR=false)	*P*(ASR=true | Gel in cracks=true)	*P*(ASR=true | Gel in cracks=false)
Original	80	15	40	3
Test 1	95 (+15%)		45 (+5%)	1 (-2%)
Test 2	60 (-20%)		33 (-7%)	9 (+6%)
Test 3		35 (+20%)	22 (-18%)	3 (0%)
Test 4		5 (-10%)	66 (+26%)	5 (+2%)

cracks=false) increasing slightly from 3% to 9%. In the opinion of the authors, neither of these changes would significantly affect a decision.

Test 3 and 4 reviewed the conditional probabilities of *Gel in cracks* when *ASR* was false. In Test 3, the *P*(*Gel in cracks*=true|*ASR*=false) was increased by 20% from 15% to 35%, implying that the gel could be present without the occurrence of damaging ASR i.e., it is not necessarily symptomatic of damaging ASR. While no effect was observed in the *P*(*ASR*=true|*Gel in cracks*=false), a decrease of 18% in the *P*(*ASR*=true|*Gel in cracks*=true) was noted. In Test 4, the expert's value for *P*(*Gel in cracks*=true|*ASR*=false) was decreased from 15% to 5%, meaning that the production of gel from other sources was very unlikely. As expected, the *P*(*ASR*=true|*Gel in cracks*=true) increased dramatically from 40% to 66%. The change crossed the 50% level to where the likelihood of *ASR*=true is greater than *ASR*=false. This is a critical change and shows that the *P*(*Gel in cracks*=true|*ASR*=false) should be verified either by the consensus of experts or by data because a change in this probability may affect the decision made by a user of the model.

Similar analysis was conducted for the remaining output nodes and only eight of the 276 conditional probabilities were identified as critical. In general, the critical conditional probabilities were associated with the false state of the output node, as observed in the ASR example.

4. Testing the diagnostic model

The model was tested on two case studies to determine how they would perform relative to the inspectors.

4.1. Case 1: Bridge on Bloor Street East over Mt. Pleasant road, Toronto, Ontario

The bridge, built in 1948, is a two span structure carrying five lanes of traffic. The structure is composed of a thin reinforced concrete deck over 17 steel girders. The wearing surface is asphaltic concrete. The deck is graded to drain from the road centerline towards the curbs and from west to east down the length of the deck. There are no deck drains on the structure. There is a paved-over expansion joint at the west end of the deck and a "U" shaped strip seal joint at the east end (Gervais 1998).

According to a condition survey completed in August 1998 (Gervais 1998), the deck soffit was in poor condition. Large spalls with exposed rebar were found on both spans of the structure. Wide cracks outlining delaminated areas were also found in several locations on both spans. Very large delaminated areas were found between some girders, and light to medium scaling was noted in some of the delaminated areas. Wide cracks with efflorescence were observed in many locations. Air void system analysis was not required as the deck was constructed prior to 1958, before air entraining was introduced. The waterproofing membrane was generally in fair to good condition. The results of the half cell survey indicated that the -0.35V threshold level for probable corrosion was exceeded for less than 10% of the carriageway. Areas where the corrosion potentials were the highest were located along the deck centerline at the east expansion joint and at various locations near the west expansion joint. Chloride ion content testing was performed on two cores and the concrete exceeded the threshold value of 0.05 % at depths 60 to 90 mm for only one of the cores.

To see what result the belief network model would give, the results of the condition survey were entered into the network as evidence. The evidence was entered in three stages. Stage 1 included all the site observations; Stage 2 added the test results; Stage 3 modified the state of the copper half cell and chloride content to a more severe state for corrosion to see how it would affect the output. shows the input variables, their states, and the output for each stage.

Stage 1 output gives a high probability for corrosion and freeze-thaw but low likelihood for ASR. In Stage 2, the probability for corrosion dropped from 91% to 66% because of the copper half cell results. According to the condition survey, only 10% of the deck area exhibited corrosion potential readings less than -0.35V, so readings >-0.2V were tested first. Freeze-thaw increased in Stage 2 because the deck is not properly air entrained. In Stage 3, the probability of corrosion increased again because the copper half-cell was changed to <-0.35V. The chloride content was increased which also increased the probability of corrosion. The conclusion of the model is that corrosion and freeze-thaw mechanisms are both acting on the structure. A practitioner confirmed this conclusion (the condition survey provided only test results and observations, not conclusions).

Table 4 Case 1 results

Variables	Stage 1	Stage 2	Stage 3
Inputs			
Spalling	True	True	True
Steel exposed	True	True	True
Scaling	True	True	True
Efflorescence	True	True	True
Condition of waterproofing overlay		Good	Good
Air-entrainment <5%		True	True
Spacing factor		> 350 μm	> 350 μm
Chain drag/hammer hollow sound		True	True
Copper half cell		> -0.2V	< -0.35V
Chloride content		0.02-0.04%	0.04-0.08%
Outputs			
Corrosion	91%	66%	99%
Freeze-thaw	78%	97%	97%
ASR	12%	12%	12%
Shrinkage	41%	34%	34%

4.2. Case 2: Bridge on Dundas Street Westbound over Kipling avenue, Etobicoke, Ontario

This single span bridge was built in 1962 and runs east/west. The concrete structure is post-tensioned voided slab with an asphalt wearing surface. There are no deck drains on the structure. There is a paved over expansion joint at both ends of the deck. A condition survey was prepared in May 1994 (Crespi 1994) and states that the deck underside was in poor condition with large damp areas, rust stains and numerous heavily stained and unstained longitudinal cracks. Analysis of the crack locations and spacing revealed that the cracks are aligned with the prestressing tendons in the bottom slab of the deck. The deck slab/sidewalk interface was heavily stained with efflorescence on both the north and south sides of the structure. The underside of the north and south sidewalks also exhibited several stained transverse cracks and several spalls. Heavily rust-stained cracks and spalls were observed on the underside of the north sidewalk.

Expansion joints were in poor condition with evidence of leakage on the components directly below. Values for air content, specific surface and spacing factor were all found to be outside accepted parameters. A uranyl acetate test revealed that only trace amounts of alkali-silica gel were present in the voids. The core revealed chloride contents below the threshold value necessary to depassivate embedded steel and permit corrosion. The results of the half-cell survey indicated that 5% of the deck had potential below -0.35V and about 88% of the deck had potential higher than –0.2V. The waterproofing system was well bonded to the deck concrete except in few spots. Light scaling was observed with spalling at expansion joints. All of the reinforcing steel inspected at the core sample

Table 5 Case 2 results

Input variables	Stage 1	Stage 2	Stage 3
Deteriorated joints	True	True	True
Spalling at joint	True	True	True
Scaling	True	True	True
Crack over rebars	True	True	True
Water runs or staining	True	True	True
Transverse cracking	True	True	True
Efflorescence	True	True	True
Spalling	True	True	True
Rust stains	True	True	True
Condition of waterproofing overlay		Good	Good
Uranyl acetate test		False	False
Covermeter testing		30-50 mm	30-50 mm
Air-entrainment <5%		True	True
Spacing factor		> 350 μm	> 350 μm
Copper half cell		> -0.2V	< -0.35V
Chloride content		0.02-0.04%	0.04-0.08%
Output variables			
Corrosion	99%	96%	100%
Freeze-thaw	77%	97%	97%
ASR	15%	9%	9%
Shrinkage	72%	72%	72%

locations was in good condition with no visible sign of corrosion. Concrete cover readings were on average 42 mm for transverse reinforcing and 52 mm for the longitudinal reinforcing.

Again, the evidence was entered to the model in three stages, as shown in Table 5. Stage 1 output gives high probability for corrosion, freeze-thaw and shrinkage. In Stage 2 the probability for corrosion decreased slightly because the copper half cell was input as > -0.2V, which indicates low probability of corrosion; however, it had only a minor effect.

Based on the condition survey, plans were made to replace the bridge. It is interesting to note that no significant change in probabilities resulted from adding evidence in Stages 2 and 3 that would have affected a decision of the deterioration forces at work in this case.

5. Summary

In both cases, the belief network model behaved as expected by the experts. The case studies are based on conditional reports which did not discuss the actual cause of the deterioration and therefore cannot be compare to the belief network results, however, the experts agreed on the diagnosis by the model. There was some concern that the probabilities might be too high on occasion, but that is something that can be dealt with in the calibration of the model over many years of data collection and probability refinement. The primary consideration is that the model is behaving appropriately.

6. Conclusions

In this study, a belief network model was developed to diagnose concrete bridge deck deterioration that combined expert knowledge with theoretical knowledge from the literature. The model was limited to four deterioration mechanisms: shrinkage, freeze-thaw, ASR and corrosion of reinforcement, however, in Ontario, the majority of the bridge decks are paved and therefore many of the deterioration symptoms that have been included in the model are not visible. One improvement to the model would be to add more testing methods, e.g., strength tests, to make the model more practical. The inclusion of other deterioration mechanisms would also generalize the model. It is important to note that this is a ‘work in progress’ that requires further study and refinement before it becomes a practical tool; however, what we have done provides the framework for future development.

The model was developed in the belief network environment because it can handle uncertainty and deterioration mechanism interactions, important aspects of diagnosing the deterioration of concrete. The major obstacle in building a belief network is acquiring the probabilities to complete the model, so the opinion of an expert was used for model development. Conditional probabilities for eight parent combinations that require refinement over time as data are collected were identified using sensitivity analysis.

The model confirmed that the belief network environment is very useful in combining knowledge types as well as combining discrete knowledge of deterioration mechanisms. The authors anticipate that this technique will be used increasingly to develop infrastructure management tools.

References

ASTM (1998), "Wear and Erosion; Metal Corrosion", C876-91, 03.02, ASTM.
Broomfield, John (1997), *Corrosion of Steel in Concrete*, E&FN SPON, London, UK.
Crespi, C (1994), "Detailed Deck Condition Survey, Dundas Street Westbound over Kipling Avenue", report submitted to Municipality of Metropolitan Toronto by Cole, Sherman & Associates Ltd., Toronto, Ontario.
Emmons, Peter (1993), *Concrete Repair and Maintenance*, R. S., Means Company, Kingston, MA.
French, S., (1986), "Combining probability distributions: A critique and annotated bibliography", Comment, *Statistical Science*, **1**, 138.
Gervais, M. and Brown, A. W. (1998), "Detailed Bridge Condition Survey, Bloor Street East over Mt. Pleasant Road, Toronto, Ontario", Report submitted to City of Toronto Transportation Department by Golder Associates, Toronto, Ontario.
Herodotos, Pentas, Avent, Richard, Gopu, Vijaya, and Rebello, Keith, (1995), "Field study of longitudinal movements in composite bridges", *Transportation Research Record, 1476*, 117-129.
Jensen, Finn V. (1996), *An Introduction to Bayesian Networks*, Springer-Verlag New York, Inc., New York.
Kay, Ted (1992), *Assessment and Renovation of Concrete Structures*, Longman Scientific & Technical, UK.
LeBeau, K. H., Wadia-Fascetti, S. J. (2000), "A fault tree model of bridge deterioration", 8th *ASCE Specialty Conference on Probabilistic Mechanics and Structural Reliability*, 1-6.
Mays, Geoff (1992), *Durability of Concrete Structures*, E&FN SPON, London.
McCabe, B., AbouRizk, S. M., and Goebel, R. G. (1998), "Belief networks for construction performance diagnostics", *J. Comput.,* in Civil Engineering, *ASCE,* USA, **12**(2), 93-100.
Morris, P. A. (1986), "Combining probability distributions: A critique and annotated bibliography", Comment, *Statistical Science,* **1**, 141-144.
Neville, A. M. (1997), *Properties of Concrete*, John Wiley & Sons, USA.
Pearl, J. (1988), *Probabilistic reasoning in intelligent systems: Networks of plausible inference,* Morgan Kaufmann Publishers, Inc., San Francisco, CA.
Poole, D. L, Mackworth, A., and Goebel, R. G. (1998), *Computational Intelligence: Logical Introduction*, Oxford University Press, New York, NY.
Sianipar, P. and Adams, T. (1997), "Fault tree model of bridge element deterioration due to interaction", *ASCE J. Infrast. Sys.*, **3**(3), 103-110.
Small, E. P, Philbin, T., Fraher, M., and Romack, G. P. (2000), "Current status of bridge maintenance system implementation in the Unites States", *8th International Bridge Management Conference*, April 1999, Denver, CO, published in TRB Transportation Research Circular 498, **1** A-1/1-16.
Stark, David (1991), *Handbook for the Identification of Alkali-Silica Reactivity in Highway Structures*, Strategic Highway Research Program, National Research Council, Washington DC.
Stiber, N. A., Small, M. J., and Pantazidou, M. (2004), "Site specific updating and aggregation of Bayesian belief network models for multiple experts" *Risk Analysis* **24**(6), 1529-1538.
Thompson, P. D., Merlo, T, Kerr, B, Cheetham, A. and Ellis R. (2000), "The new Ontario bridge maintenance system", *8th International Bridge Management Conference*, April 1999, Denver, CO, published in TRB Transportation Research Circular 498, **2**, F-6/1-15.
Wallbank, E. J. (1989), *The Performance of Concrete in Bridges*, G. Maunsell & Partners, London.

CM