Reliability Assessment of Temporary Structures Using Past Performance Information

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Abstract: Temporary structures are structural systems set up for applications in relatively short time periods for such functions as maintenance, repair or retrofit of a structure or for staged performances. Examples include scaffolds, temporary shelters, temporary walkways, tents, temporary supports, and other facilities used for a limited service time. Design of temporary structures to dead and live load does not often impose any particular difficulty, however design to seismic or wind load requires a more careful investigation. This is because the service life expected of a temporary structure is much shorter than a "permanent structure," and the probability of extreme load exposure to the temporary structure is substantially less. Thus it will make sense to use a reduced load level proportional to the intended service life of the temporary structure. Using a reduced load level may be a reasonable assumption for a structure that is only used once. However, the decision to allow the same structure to be reused brings about certain ambiguity as to what happens to the level of risk inherent in the structure, which is now subject to repeated use and some reduction in its capacity due to periodic disassembling and reassembling. This paper provides an overview of risk analysis models for temporary structures (by the lead author and his coworkers). In an attempt to provide a decision-making strategy for repeated usage of a given temporary structure, the performance record of a structure is used in predicting its future condition in the form of a probability of failure. This probability of failure is demonstrated as being a key factor in the decision-making process. To quantify the model, the special case of tube and coupler scaffolds is investigated. Considering stability requirement for a scaffold, gross capacity values are obtained for several configuration types. These capacity values are also important in computing the probability of failure of the systems and decision-making in whether to allow a system to continue with its usage, as explained in the paper.

Keywords: Reliability; Scaffolds; Seismic Forces; Temporary Structures; Wind.

1. Introduction

Temporary structures are structural systems set up for applications in relatively short time periods for such functions as maintenance, repair or retrofit of a structure or for staged performances. Examples include scaffolds, temporary shelters, temporary walkways, tents, temporary supports, and other facilities used for a limited service time. In repair and retrofit applications, scaffolds (whether of the supported or suspended types) are often attached to a host structure and will need to be designed with considerations for (1) the design requirement of the host structure; and (2) the time period during which the scaffold is set up.

Design of temporary structures to dead and live load does not often impose any particular difficulty, however design to seismic or wind load requires a more careful investigation. This is because the service life expected of a temporary structure is much shorter than a "permanent structure;" and, as a result, the probability of extreme load exposure to the temporary structure is substantially less. Thus, it will make sense to use a reduced load level proportional to the intended service life of the temporary structure as has also been suggested by Rosowsky (1998) and Mohammadi and Zamani (2008). However, the short service life does not imply less importance. Most recently there were several catastrophic failures of temporary structures, which were intended for outdoor performances, in August 2011 – one in Indianapolis, Indiana and another in Belgium. Both failed as a result of severe wind effects. On May 4th, 2014, mechanical equipment was blamed for a stage failure during a circus performance in Providence, Rhode Island. The March 2011 earthquake in Tohoku, Japan caused substantial damage to many critical facilities. These facilities will require using repair platforms and scaffolds in which their design for seismic activities is important and must carefully be done. These incidences point to the importance of temporary structures and their requirement for proper design.

To address the importance of temporary structures in design, two issues will need to be considered. One is the design load; and the other being the decision-making to allow repeated usage. As described earlier, the design load can be obtained based on the lower exposure risk. This often results in a lower value than is expected in a permanent structure. Using a reduced load level may be a reasonable assumption for a structure that is only used once. However, the decision to allow the same structure to be reused brings about certain ambiguity as to what happens to the level of risk inherent in the structure, which is now subject to repeated use and some reduction in its capacity due to periodic disassembling and reassembling (Mohammadi, 2006; Mohammadi and Zamani, 2008). The decision to whether continue or discontinue with the usage can be made by estimating the risk of failure of the temporary structure with considerations for its geometry, its resistance to withstand load and the conditions to which the structure has been exposed to. This is to say whether the structure has experienced any loading condition that may have comprised its overall capacity and thus affecting its risk of failure.

The computation of the risk of failure of a temporary structure to seismic and wind load environment has been suggested by Mohammadi (2006) and Mohammadi and Zamani (2008). In these papers, a Bayesian updating process was adopted to use the performance record of a structure in a past usage period in estimating the risk of failure in a subsequent period. This risk level was then suggested as a means for decision making in whether to continue or discontinue the repeated usage of the structure. This paper provides the underlying concept for the risk prediction and outlines the method used to adjust the risk of failure based on past performance records. Furthermore, to quantify the model, the special case of tube and coupler scaffolds is investigated (Zare Najafabadi, 2012). Considering stability requirement for a scaffold, overall capacity values are obtained for several configuration types for scaffolds and with considerations for the geometrical conditions of joints, attachment to the host structure, and load exposure. A scaffold's modes of failure considered include (1) potential overstress in vertical and diagonal members; and (2) stability due to sliding (loss of friction resisting the lateral movement) or overturning. The overall capacity values obtained are also important in computing the probability of failure of the systems and decision-making in whether to allow a system to continue with its usage, as explained in the paper. The details of the analyses conducted for the quantification of the scaffold capacity are provided and discussed in the paper along with illustrative examples.

2. Background

Temporary structures rarely receive the attention of structural engineers unless their failures result in great damages, injuries and loss of lives. A review of several recent failure cases that have resulted in significant damage indicates that the cause of failure can be attributed to one or more of the following factors.
(1) Inadequate design of system components;
(2) Inadequate prediction of the type of load prevalent;
(3) Underestimation of the system capacity to withstand loads; and,
(4) Natural load effects exceeding the design criteria (this factor was specifically mentioned in at least four major failures in recent years – two of which occurred one in Indianapolis, Indiana and another in Belgium in 2011 as a result of collapse of stages set up for performing art. However, equally likely, seismic events can also contribute to the failure when rare occurrences cause overstresses or instability in the system).

As indicated earlier, two important issues will need to be addressed in design of a temporary structure. These are (1) the reduced design load; and (2) the decision-making process whether to continue or discontinue the usage. Temporary structures are often designed for a lower load level than those used in permanent structures. Thus, unless there is a flaw in structural components and connections of a temporary structure, the incidences of failure, when the load exceeds the design values, are generally considered as chance occurrences that can be predicted only through probability values. These probability values are crucial in accepting a risk level in design and in the decision-making process in whether to continue or discontinue the use of a temporary structure or whether additional strengthening may be required in their design. The reduced design load levels have been rather extensively studied and also have been implemented, to a certain extent, in recommended practices (e.g., SEI/ASCE 37-02 document, ASCE, 2002; CALTRANS, 2004; Rosowsky, 1998; Ratay, 2004). However, the issue related to the decision making in regard to the repeated usage has not been as well addressed. As indicated, this issue needs formulations through probabilistic studies. In a related study, Boggs and Peterka (1992) use a reliability framework to arrive at design wind speeds for temporary structures. The design wind speed proposed is based on the shorter exposure time expected for temporary structures. The probability of failure from wind loads is estimated in terms of such parameters as the wind speed probability distribution (extreme value type I distribution), factor of safety, mean recurrence interval of the design wind speed, and the exposure period. An illustrative example is provided to demonstrate how their method based on the reliability formulation can work. The design wind velocity in the example cited (for a factor of safety of 2 and a 2-year exposure time) is obtained to be a reduced value of 53 mph with a 0.5 percent probability of failure within the exposure time. Hill (2004) provides a comprehensive discussion on loads for temporary structures and offers a critique on the method presented by Boggs and Peterka. Hill suggests that in certain cases the use of reduced load (derived on the basis of a shorter life expected for a temporary structure) may be justified. An example is given for a case of temporary tent. Hill then presents examples for which the reduced load approach may not necessarily offer a viable method. This is reported to be especially true when a structure has already gone through its short service life. If the decision is to continue the usage for several more years beyond the structure's short life, then the designer would face a decision to make as to whether to allow the continuation of usage of this structure, while in reality it has been only designed for a reduced load effect.

Mohammadi (2006) discusses Hill's argument further and proposes to use the performance record of a given system (during its first intended service life) as a basis to modify the system risk to extreme load exposure. Mohammadi then argues that this modified risk may be used as a parameter in decision-making process as to whether allow the structure to continue its service into a second usage cycle. On a follow up study to further investigate the significance of the performance record of temporary structures as a means to modify their risk of exposure to extreme events, Mohammadi and Zamani (2008) report on a method using the Bayesian updating approach to modify the risk of a temporary structure to wind and earthquake loads. The study was limited to basic development of an updating model and utilizing various levels of wind velocity and earthquake ground acceleration intensities as a means to define the resisting capacity of the structure.

An overview of the use of past performance record as means to estimate the risk is explained below. In any given period of usage, the performance of the structure may be explained as two possible conditions: (1) The temporary structure did not experience any major load during its usage; and (2) the temporary structure experienced a major load during its first usage and survived.

As explained in Mohammadi and Zamani (2008), with the first situation, the structure performed well and passed its original intended short service life. A new (modified) risk level is obtained based on the fact that it did not experience a major load. This is considered as a performance record and can be used along with a Bayesian approach (Ang and Tang 2007) in modifying the risk of failure of the structure. The Bayesian approach is employed to modify the occurrence rate of the extreme event (e.g., wind load) as described below.

As explained in Mohammadi and Zamani (2008), if θ is the occurrence rate for the extreme load event, the data on θ is updated sing the following equation:

$$P''(\theta = \theta_i) = \frac{P(\varepsilon \mid \theta = \theta_i) P'(\theta = \theta_i)}{\sum_{i=1}^{n} P(\varepsilon \mid \theta = \theta_i) P'(\theta = \theta_i)} \tag{1}$$

where

$P'(\theta = \theta_i)$ = prior probability that the occurrence rate is equal to θ_i based on prior statistical data.

$P''(\theta = \theta_i)$ = posterior probability that the occurrence rate is equal to θ_i based on new information.

ε = outcome of any new information. In the case of a temporary structure this outcome is directly related to the structure's performance record in a previous usage period.

n = the number of possible values for the parameter θ.

Statistics of the parameter θ depend directly on any occurrences of the applied load during the previous usage period of the structure. Mohammadi and Zamani (2008) clarify this as follows. Let's assume the original design was for a 50-year strong wind and the first intended service life of the structure was two years. Furthermore, let's assume during this period no severe wind loads (in excess of the design load) occurred. In Eq. (1), the updating information is described with: "no occurrences within two years." Thus considering two possible values for the occurrence rate of strong winds, one possibility is zero occurrence (i.e. θ_1 = 0) for two years and this corresponds to only 2/50=0.04 probability; because it concerns only 2 years of the 50-year period.

A second possibility for the occurrence rate is 1/50=0.02 (based on the original design value equal to the 50-year event); and this value (i.e. θ_2 = 0.02) has a probability equal to 1 – 0.04 =0.96. The 0.04 and 0.96 probability values are prior probabilities. Since there was no occurrence of the extreme event in the first two years, assuming for simplicity that these extreme events follow a Poisson distribution model in time (Khisty, Mohammadi and Amedkudzi, 2012), the new outcome (ε) can be defined as ε = (X=0 in t=2 years); and from Poisson distribution

$$P(\varepsilon \mid \theta = \theta_1) = P(X = 0 \mid \theta = \theta_1) = e^{-\theta_1 t} = e^{-(0\times 2)} = 1$$
$$P(\varepsilon \mid \theta = \theta_2) = P(X = 0 \mid \theta = \theta_2) = e^{-\theta_2 t} = e^{-(0.02\times 2)} = 0.9608$$

Now applying the Bayesian updating equation, the following posterior probabilities are computed

$P''(\theta = \theta_1)$=0.0416 and $P''(\theta = \theta_2)$=0.9584

The new statistics for the return period can now be used in modifying the risk in the decision making process in allowing the structure to continue into a second usage cycle. If the temporary structure experienced a major load application and survived, a similar formulation can be used in updating the occurrence rate. However, in this case, it is noted that there was one occurrence during the first 2 years of service of the structure. This means that θ_1= 0.5 per year (i.e., one occurrence in two years); and again this corresponds to only 2/50=0.04 probability. A second possibility for the occurrence rate is again 1/50=0.02 (i.e. θ_2 = 0.02) with a probability equal to 0.96. The new outcome (ε) in this case can be defined as ε = (X=1 in t=2 years); and using the updating process the posterior probabilities are

$P''(\theta = \theta_1)$=0.2853 and $P''(\theta = \theta_2)$=0.7147

As explained by Mohammadi and Zamani (2008), in this case, there was a load application and the structure survived. This piece of information can also be used in modifying the distribution model for the resisting capacity of the structure. This would be similar to the process that is used in "proof load testing" of structures (see for example Fujino and Lind, 1977). If the resisting capacity of the structure corresponding to this applied load is r_1, then all possible values for the resistance that are smaller than r_1 no longer exist in the resisting capacity distribution model. Considering the original probability density function of the resisting capacity be $f_R(r)$ and the new (updated) function as $f'_R(r)$, then

$$f'_R(r) = \frac{f_R(r)}{1 - F_R(r_1)} \tag{2}$$

in which $F_R(r_1) = \int_{-\infty}^{r_1} f_R(r)dr$.

The modified risk can now be obtained based on the updated information on the occurrence rate and the resisting capacity probability model. The process can be repeated for multiple usage of the structure. After each usage period, based on the outcome of the extreme load activities, the risk can be modified for the next usage cycle and be used in deciding whether the risk level is still acceptable for the structure to be used again.

3. Specific Case of Scaffolds

As seen in the model presented above, the information on the load and resistance for the temporary structures is needed in arriving at a risk level. The probability distributions for the wind load can be based on established extreme value modes. For seismic loads, some type of seismic risk model (which is also well-established) or information and hazard maps available through the United States Geological Survey (USGS) sites can be used. However, the resistance (R) is structure-specific and will need to be determined. More specifically, a single random variable describing an overall resistance of the temporary structure is needed. In a realistic case, R will be written as a function g of several random variables R_i, i.e.

$$R = g(R_1, R_2, \ldots) \tag{3}$$

It is noted that most temporary structures are simple systems; and as such, one may be able to arrive at an estimate for R without dependence on complicated probabilistic formulations that require inter-correlation data between R_i components. For example, assuming all R_i components are perfectly correlated, R can be represented by the resistance of the component that has the highest probability of failure. To further demonstrate the quantification of the model with specific data on the resistance, the special case of tube and coupler scaffolds was considered (Zare Najafabadi, 2012). These systems are made up of tubular structural elements interconnected through simple means and braced to form one, two or more story systems for structural repair (see Fig. 1).

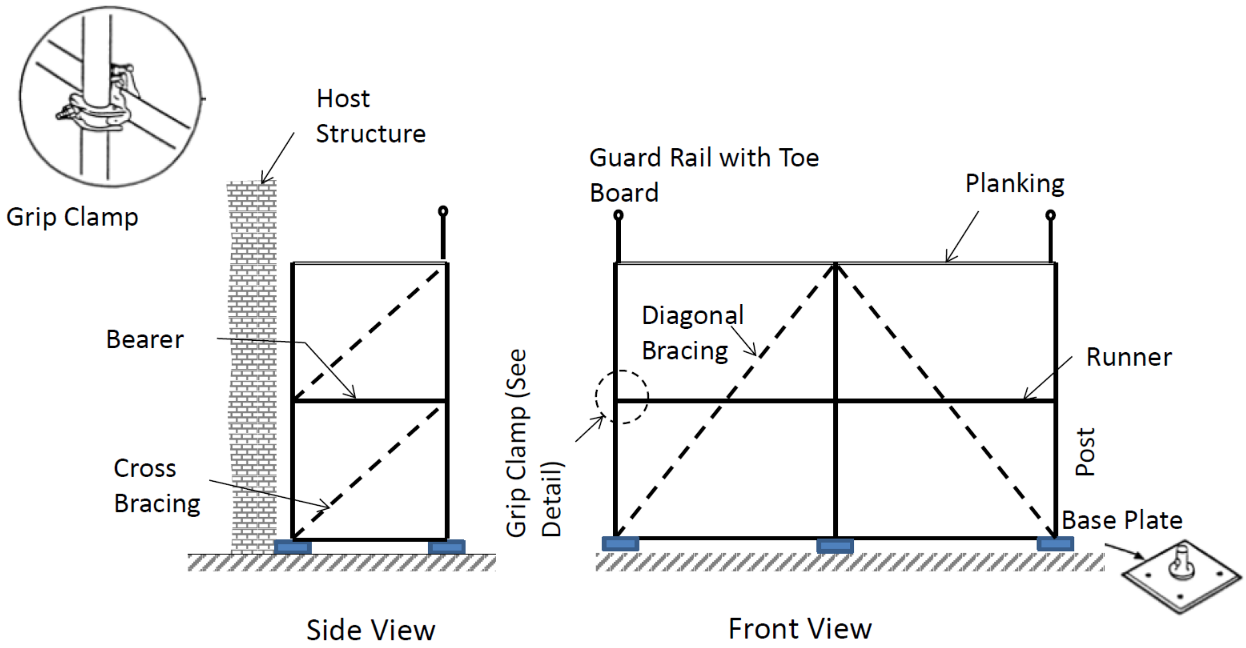

Figure 1. Schematics of a tube and coupler scaffold

The Occupational Safety and Health Administration (OSHA) of the United States Department of Labor sets requirements on use of tube and coupler supported scaffolding systems (OSHA, 2012). According to these requirements, the maximum height of such scaffolds is limited to 125 feet (38.1 meters) unless designed by a registered professional engineer and constructed and loaded in accordance with that design. Further requirements are defined based on the load category of scaffolds: namely, light duty, medium duty, and heavy duty. These categories are defined by the maximum intended working load that the scaffold will withstand. For light duty scaffolds, the maximum working load is limited to 25 pounds per square foot (1,200 Newtons per square meter). For medium duty scaffolds, the maximum working load is limited to 50 pounds per square foot (2,400 Newtons per square meter); and for heavy duty ones, the maximum working load is limited to 75 pounds per square foot (3,600 Newtons per square meter).

Per definition, the working load includes the load by workers, materials, and equipment imposed on the working deck of the scaffold.

For light duty tube and coupler scaffolds, the size of structural members is prescribed by the diameter and the wall thickness of the circular tubes used. Generally, about 2 inches (nominal size which is 1.9 inches in actual) is suggested for the outside diameter of the steel circular tube for applications of posts, runners, and braces. The same size element is allowed to be used for bearers with the additional requirement of maximum post spacing of 4 feet by 10 feet (1.2x3.0 m). OSHA also bounds the maximum vertical spacing between runners to 6 feet and 6 inches (2 m) for light duty type of scaffolds.

For medium duty tube and coupler scaffolds, similar to the light duty type, a nominal 2 inches (1.9 inches) outside diameter (OD) steel tube or pipe for applications of posts, runners, and braces are permitted. It provides two size options for bearers, namely a nominal 2 inches (1.9 inches) OD steel tube or pipe for a maximum post spacing of 4 feet by 7 feet (1.2x2.1 m) or Nominal 2½ inches (2.375 inches) OD steel tube or pipe for a maximum post spacing of 6 feet by 8 feet (1.8x2.4 m). Maximum vertical spacing between runners is the same as those posed on the case of light duty scaffolds (OSHA, 2012).

For heavy-duty tube and coupler scaffolds, similar to the light and medium duty type, a nominal 2 inches (1.9 inches) outside diameter (OD) steel tube or pipe for applications of posts, runners, and braces are permitted. The size for the Bearers is specified to be a nominal 2½ inches (2.375 inches) OD steel tube or pipe for a maximum post spacing of 6 feet by 6 feet (1.8x1.8 m). Similar maximum vertical spacing between runners to those required for light and medium duty scaffolds is required for these types also.

In addition to the aforementioned design requirements, several requirements for tube and coupler scaffolds are also specified as described below:

(1) Transverse bracing as an X shape across the width of the scaffold will need to be installed at the scaffold ends and at least at every third set of posts horizontally and every fourth runner vertically;

(2) Bracing will need to be extended upward diagonally to opposite sides of the scaffold;

(3) Longitudinal bracing will need to be installed at least every fifth post when length of the scaffold is larger than its height; and,

(4) Longitudinal bracing will need to be installed from the base of the end posts upward to the opposite end posts and continue in alternating direction until reaching the top of the scaffold when length of the scaffold is smaller than its height [1].

Moreover, OSHA requires that scaffolds and their components shall be capable of supporting four times of their maximum intended loads without failure. Considering the seismic effect, the scaffold has two modes of failure. These are (1) failure due to overstress at structural components; and (2) failure because of instability. As a result of ground shaking, the instability occurs when a horizontal sliding compromises the resisting friction force at the base plates.

In the study conducted by the authors, various configurations for the scaffolds (in terms of the number of stories) were considered; and in all cases, the instability was observed as the dominant mode of failure. Also, it is noted that the maximum spacing between tube and coupler scaffolding posts was limited to 4 feet (1.2m) by 7 feet (2.1m) for scaffolds with 2 inches nominal OD bearer or 6 feet (1.8 m) by 8 feet (2.4 m) for scaffolds with 2½ inches nominal OD bearer. Furthermore, the maximum vertical spacing for the runners was limited to 6 feet and 6 inches (2.0m) as required by OSHA. The analyses include a finite element modeling of each scaffold with appropriate boundary conditions to simulate the frictions at the base plates and the free movement and rotations at grip joints (Figure 1 presents a typical two-bay, two-story scaffold used in the analysis). The input motion for seismic included the 1940 El Centro Earthquake record adjusted by using multiplies representing fractions of up to 25% of the accelerations in the record.

Columns 1 and 2 of Table 1 provide typical results indicating the earthquake acceleration level at which the scaffold will be subject to instability. For example, for a 1-story, 1-bay (or 2-bay) system, the failure occurs at a peak acceleration of about 0.06g, which represents the overall resistance of the scaffold (Zare Najafabadi, 2012).

Table I. System overall resistance

Scaffold Configuration	Acceleration Level (R) (in g's)	p = P($R \le S$), Mean S = 0.025g	p_f (for 5 years)
1-bay, 1 story	0.060	0.0062	6.2E-05
1-bay, 2-story	0.030	0.4105	0.0041
2-bay, 1-story	0.008	0.9994	0.0099
2-bay, 2-story	0.007	0.9999	0.0099
3-bay, 1-story	0.012	0.9820	0.0098
3-bay, 2-story	0.012	0.9820	0.0098

As seen in Table 1, the 3-bay system had a higher resistance than the one with two bays. This is because of a slight increase in the dead weight of the system which resulted in a higher friction force at the base plates.

Information such as those in Table 1 can be used to establish a probability density function for the resistance R and modify the function if the system has experienced an applied load (acceleration) of r_1.

4. Computing Risk of Failure

The scaffolds analyzed in the study are typical of those used in utility companies; and as such they serve critical structures. For a site located in Northern Illinois, the USGS data indicates a peak acceleration of about 0.025g with 10% probability in 50 years (i.e., a 500-year return period), which can be approximately considered as the activity rate using a Poisson distribution for the occurrence of earthquake events. If we accept this acceleration level as the mean applied load (S), and for a 500-year period, the probability of S exceeding R can be computed using the following equation

$$p = \int [1 - F_S(r)] f_R(r) dr \tag{4}$$

in which, p = P(R≤S), $F_S(s)$ = probability distribution function for the applied load and $f_R(r)$ =probability density function of the resistance. Using lognormal probability density functions for the resistance (R) and the applied load with each random variable having a coefficient of variation of 0.25, p can be computed for the six scaffold configurations. The results are shown in Column 3 of Table 1. In computing p, the values of R listed in Table 1 are considered as mean values of the resistance.

Equation 4 describes the risk of failure if there is certainty that the extreme event (earthquake) occurs. Since in reality, there is also a probability associated with these extreme events, in a time period t, the probability of occurrence of the extreme events can be described as $P(X=n)$, in which $X=n$ describes occurrence of n extreme events in time t. Incorporating this probability, the risk of failure of the structure is

$$p_f = \sum_{n=0}^{\infty}[1-(1-p)^n]P(X=n) \tag{5}$$

If the a Poisson distribution (Khisty, Mohammadi and Amedkudzi 2012) is used in describing the occurrence of extreme events, Eq. 5 is written as

$$p_f = \sum_{n=0}^{\infty}[1-(1-p)^n]\frac{(\theta t)^n}{n!}e^{-\theta t} = 1-e^{-\theta p t} \tag{6}$$

For the purpose of illustration, we assume a scaffold is planned to be standing for 5 years. Using Eq. 6 and an activity rate equal to θ = 1/500, the values for the probability of failure are computed and recorded in the fourth column of Table 1. As seen in Table 1, some structures will have relatively large probability of failure and as such may have to be redesigned to reduce their risk of failure. This may require securing a scaffold to the host structure.

Updating the occurrence rate – When the occurrence rate θ is updated after using the structure for one usage period, new values of θ are then used to find p_f. If there are only two possible values for θ, each with posterior probabilities computed as $P''(\theta=\theta_1)$ and $P''(\theta=\theta_2)$, then p_f in Eq. 6 becomes

$$p_f = [1-e^{-\theta_1 p t}]P''(\theta=\theta_1)+[1-e^{-\theta_2 p t}]P''(\theta=\theta_2) \tag{7}$$

Mohammadi and Zamani define p_f as the "modified risk." The modified risk is compared with the original risk of failure inherent in the design of the structure and is used to decide whether one must continue with the usage of the temporary structure into a next usage cycle. Now assume during the first usage period an earthquake occurred and caused a small level of acceleration in a scaffold. However, initial inspection revealed no damage to the system. This is considered as performance data and can be used in updating the activity rate. We interpret this information as follows:

(1) There are now two possibilities for the activity rate, θ_1=1/5 =0.2 and θ_2=1/500=0.002.

(2) The probabilities associated with these activities are estimated as distribution
$P'(\theta=\theta_1)$=5/500 =0.001 and $P'(\theta=\theta_2)$=1 – 0.001=0.999

(3) The event occurred in the first 5 years is explained via the outcome ε (event of an earthquake in 5years); and $P(\varepsilon\,|\,\theta=\theta_1)=P(X=1\,|\,\theta=\theta_1)=\theta_1 t e^{-\theta_1 t}=0.2\times 5e^{-(0.2\times 5)}=0.36788$
$P(\varepsilon\,|\,\theta=\theta_2)=P(X=1\,|\,\theta=\theta_2)=\theta_2 t e^{-\theta_2 t}=0.002\times 5e^{-(0.002\times 5)}=0.00990$

(4) Now applying the Bayesian updating (Eq. 1), the following posterior probabilities are computed
$P''(\theta=\theta_1)$=0.0360 and $P''(\theta=\theta_2)$=0.9640.

The new statistics for the activity rates can now be used in modifying the risk of failure of the scaffold for the next usage period (5 years). Using Eq. 7, the risk of failure in 5 years when $p = 0.0062$ is computed as $p_f = 0.000283$. This is compared with $p_f = 6.2\text{E}–05$. The result indicates that the risk is now increased substantially (6.2E–05 /0.000282 = 46 times). This increase in the risk may substantiate a necessary action on whether to discontinue the usage of the scaffold in the second period or provide some type of strengthening to make it more stable for application in the next 5 years.

5. Conclusions

The following are the main conclusions from this study:
(1) Past performance records can be used in a Bayesian updating process to modify the risk of failure of a temporary structure in seismic or wind load environments.
(2) The modified risk can be used as a key decision-making parameter in deciding whether to continue or discontinue the use of a temporary structure from one usage period to the next.

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