

Economic and Weibull Analysis of High Performance Concrete Pavements

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ABSTRACT

This paper examines the 60-year design philosophy, specifications, and institutional maintenance and repair strategies for Minnesota's high performance concrete pavements in the first 15 years of service life. It catalogues various modifications and improvements made to the specification since the inception of the concept. It performs a life-cycle cost analysis on the original segment and an incremental benefit cost analysis. Results indicated that the high performance concrete option had the lowest Net Present Cost of investment in comparison to other competing options thus validating the 2001 choice of the high performance concrete pavement over whitetopping, bituminous pavement and do-nothing options. In a conservative approach, Waddoli Weibull analysis of Mean-Time-Between-Maintenance-Activities (MTBMA) of more than 200 concrete test sections in the network indicated that most of the sections exhibited random failure modes in the first three rehabilitation/ maintenance intervals but showed end of life ("wear-out") failure mode after the fourth rehabilitation / maintenance. Consequently, the process also facilitated the determination of optimal intervals for the four interventions within the agency-instituted 50 year analysis period.

Keywords: High Performance Concrete, Incremental benefit cost, Weibull distribution, Characteristic Life, Threshold Time, Shape Function.

INTRODUCTION

Minnesota Department of Transportation (MnDOT) recognized the importance of providing continuously flowing traffic in conformance to the vision of safe affordable and efficient transportation system extant in the mission of providing an efficient movement of people goods and services from one place to the other. It also recognizes that meeting these provisions necessitates prudence with initial costs as well as net present worth of investments in intervention associated with these initiatives. This paper examined the user cost implicit in the regular use of concrete pavements and ascertained that if the design is for 60-years it may minimize the frequency of lane closure and maintenance or rehabilitation within the service life of the pavement. Life cycle cost analysis and incremental benefit cost analysis were performed. To evaluate the efficacy of institutional maintenance / rehabilitation intervals, mean time before maintenance activity (MTBMA) was analyzed for the first second and third intervals using the multivariable Waloddi Weibull analysis. Portland cement concrete (PCC) transverse joint design, drainage design, and shoulder type, significantly affect pavement performance and must be considered during the pavement structural design process and in evaluation of alternatives within the justification process. The 60-year concrete also known as High Performance Concrete (HPC) pavement design initiatives were thus conceived as a means of providing uninterrupted flow of traffic through a pavement system designed for minimum maintenance within a 60-year period. This program recognized through projected performance that proper materials selection, design and construction practices

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were indispensable. Materials and processes have been developed and refined (Table 1). Currently, MnDOT operates a schedule that basically performs a joint reseal and Minor Concrete pavement rehabilitation in the 17th year. At the 27th year, minor concrete pavement rehabilitation and some full depth repairs are performed and at the 40th year, major concrete pavement rehabilitation is performed with full depth repairs.

Burnham Izevbekhai and Rangaraju (2006) examined the 5-year performance of the original Interstate Highway 35W 60-year design. From cores taken in the test section, and from ride measurement conducted as well as visual observations for pop outs in the section or excessive joint deterioration, it was evident that some existing distresses could have been averted by preventive maintenance. A visual survey of the area in 2013 in the section also showed that the joints have remained in good condition after 15 years with a few sliver spalls and some scattered patching. TH 100 in Robbinsdale another 60-Year design segment was completely void of joint deterioration after 13 years. The 5-year performance paper indicated a very insignificant change in ride quality after 5 years. Annual IRI measurements in the test section also indicated that the performance was still in good condition and similar to the initial post construction performance. Petrographic analysis showed no evidence of deleterious aggregate that could ultimately expand and weaken the concrete.

BENEFITS OF THIS RESEARCH & PAPER

Many years into the deployment of the 60 year concrete pavement design initiative, many practitioners had enquired if an incremental benefit cost evaluation was ever done. They also enquired if and if retro actively the high-performance concrete has exhibited performance characteristics validating their choice and initial projections. Although none of the 60-year designs has been in service for 60 years, current performance characteristics at the 15th year facilitate more tenable projections than at the first year. This research evaluates pavements at their 15th year and retroactively performs life cycle analysis as well as incremental benefit cost analysis. It also evaluates the failure characteristics of concrete pavements of conventional designs in the network in a conservative analysis of the high performance concrete.

Finally the paper accentuates that after 15 years of performance of known test sections, the advantages of the 60-year design pavement are validated.

USAGE AND PROGRESSION OF 60-YEAR CONCRETE PAVEMENT INITIATIVE

MnDOT adopted HPC as the standard design for high-volume, urban highways in 2000. Although HPC has been successfully implemented in a number of projects within Minnesota, MnDOT continues to investigate explore validation of expected service life. MnDOT's concerns include concrete durability and construction-related issues. Current materials specifications used in the design of HPC rely on accelerated laboratory testing. There were initial concern whether the permeability and air content systems in the HPC will be able to provide the necessary level of durability for the entire design period under the State's extreme climate conditions. MnDOT has also experienced some construction challenges from the use of new materials. Most HPC projects include supplementary cementitious material (SCM) to meet the concrete mixture requirements. Many contractors who were not familiar with the HPC concrete mixtures containing SCM have complained of difficulties in handling these mixtures. Table 1 shows the changes that had occurred in the specs since inception of HPC in Minnesota. Training of the engineers and contractors will be critical for continuous success of HPC in the future.

Table 1. Specification Changes

Item	Standard (2006)	2013 Changes
Design Life	Design long-life concrete pavements (HPCs): 60-years.	No Change but requires 50 Yr. LCCA of alternatives
Cross section	Slab thickness: 290 to 340 mm (11.5 to 13.5 in.), depending on truck traffic. Base: 75 to 200 mm (3 to 8 in.) dense-graded granular base (MnDOT CL-5 material) or 125 mm (5 in.) open-graded aggregate base on top of 100 mm (4 in.) CL-5. Subbase: 300 to 1200 mm (12 to 48 in.) select granular (frost-resistant) subbase.	No specified thickness but dependent on design. Base and Subbase requirements are unchanged
Joint Design	Joint spacing: 4.6 m (15 ft.).All transverse joints are doweled.	No change
Dowel Bar	Diameter: 38 to 45 mm (38 mm typical) (1.50 to 1.75 in. [1.50 in. typical]). Length: 380 to 450 mm (380 mm typical) (15 to 18 in. [15 in. typical]). Spacing: 300 mm (12 in.). Material: must be corrosion-resistant (stainless steel solid, clad, tube; plastic-coated steel; zinc-clad).	Diameter:1.50 in. for 10 ½ - 14 in. pavement thickness. Length: 380 mm (15 in) glass fiber reinforced polymer, steel tubes and zinc coated dowels added
Surface Texture	AstroTurf or broom drag. Requires 1 mm (0.04 in.) average depth in sand patch test (ASTM E 965). Transverse tining is not used due to noise	No Change
Alkali-Silica Reactivity (ASR)	Fine aggregates requires test for ASR potential by ASTM C 1260. Expansion to be 0.15% or less. Reject if the expansion is greater than 0.3%. Mitigation is required by using GGBFS or class C fly ash when the expansion is between 0.15 and 0.30%: 15 to 25%: GGBFS 35% or fly ash 20%. 0.25 to 0.30%: GGBFS 35% or fly ash 30%.	Class C deleted. GGBFS 35% or fly ash 30% in accordance with C-3115, “, Fly Ash modified with at least 66.0% SiO ₂ + Fe ₂ O ₃ on a dry weight basis and at least 38.0% SiO ₂ .
Aggregate Gradation	Combined gradation based on 8-to-18 specification: percentage retained in all specified sieves should be between 8 and 18%, except finer than no. 30, and the coarsest sieve.	Optional Incentive: Combined gradation based on 8-to-18 or 7-to-18 specification: (percentage retained in all specified sieves should be between 8 or 7 and 18%, except finer than no. 30, and the coarsest sieve) Or combined gradation based on Gradation Zone II-A of the Coarseness Factor Chart: using the coarseness factor and workability factor.
Concrete Permeability	Use of supplementary cementitious materials (GGBFS or class C fly ash) is required to lower the permeability of concrete. Specification requires rapid chloride ion permeability test value of 2500 coulomb or	RCP permeability requirement deleted

	less at 28 days, by ASTM C 1202.	
Air Content	HPC concrete mixtures: $7.0 \pm 1.5\%$. Increased air content for possible loss of entrained air due to over-vibration or in-filling with secondary compounds at later ages.	No Change
w/cm	0.40 or less.	An incentive for less than 0.4 w/c ratio added
Curing	A poly-alpha-methylstyrene membrane cure is used under normal weather conditions. No construction or general public traffic is allowed for 7 days or until the flexural strength of concrete reaches 2.4 MPa (350 lb/in ²).	Linseed oil alternative added. No construction or general public traffic is allowed for 7 days or until the flexural strength of concrete reaches 2.4 MPa (350 lb/in ²) or until compressive strength of concrete reaches 20.6 MPa (3000 lb/in ²).
Const. Quality	Requires monitoring the vibrators during paving. Paver track speed and vibrator operating frequencies must be researched daily. Initial Profile Index values, using 5-mm (0.2-in.) blanking band, greater than 126 mm/km (8 in/mi) require diamond grinding or similar action.	Initial PI requirements with 0.2 Blanking band replaced with IRI and incentive/ disincentive clause
Sealant	Elastomeric (Preformed) joints	No Change

MnDOT = Minnesota Department of Transportation, ASTM = American Society for Testing and Materials, GGBFS = ground granulated blast furnace slag, w/cm = water to cementitious material ratio

ECONOMIC & INFRASTRUCTURAL JUSTIFICATION OF 60-YEAR DESIGN MINNESOTA TRUNK HIGHWAY 100

The process for choosing a 60-year design alternative over a rehabilitation, unbonded-overlay and / or white topping alternatives was based on life-cycle-cost analysis. This method assumed a constant service level allowing the same million vehicles per day. The differences in the alternatives arose from the different repair and maintenance activities that are associated with each alternative. A net present cost of the alternatives is therefore computed through a 50 year analysis period using a discount rate of 4 %. Discount rate is the difference between the interest rate and the inflation rate, which gives a value to money if saved rather than spent. Net present value (NPV) is obtained when the benefits and costs over an alternative's life cycle are discounted to the present (Muench et al 2006), and the costs are subtracted from the benefits to yield a true NPV. If benefits exceed costs, the NPV is positive, implying project viability. If the benefits are assumed to be equal for each alternative then the net present costs are compared. Where two or more alternatives for a project exist, the one with the highest NPV over an equivalent analysis period should be a worthy investment. Policy issues, perceived risk, and funding availability or corridor-wide uniformity, however, may lead to the selection of an alternative with a lower, positive NPV (Tayabji & Lim 2006). In this analysis, NPV was the value obtained from cost alone as benefit was treated as same for the various options . Benefits include accident reduction in the section, number of vehicle miles conducted separately to realize a tenable benefit/cost ratio.

$$PV_i = \frac{1}{(1+r)^t} A_{(t)} \quad (1)$$

Moreover,

$$NPV = \sum_{i=1}^T \left(\frac{1}{(1+r)^t} A_{(t)i} \right) \quad (2)$$

where

PV = Present value at time zero (the base year);

r = Discount rate;

t = Time (year); and

A = Amount of benefit or cost in year t .

NPV = Net present value

T = Analysis period (years) and i represents the term as in first year ($i=1$) or second year ($i=2$)

Unit costs are used along with a discount rate of 4%. A discount rate is the factor used to identify a present value of goods or services when dealing with a future cost expressed in constant dollars. Discount rate selection is a critical process in life cycle cost analysis (LCCA) and directly impacts choice or implication a-priori of a fix. Most states use a set value of 4.0% for their discount rate. Minnesota Project SP 2735-159 (Izevbekhai B.I. 2001) was conceived as a reconstruction of a segment of Minnesota Trunk Highway 100 in Robbinsdale. Due to alignment changes, the reconstruction project entailed soil correction grading and base and construction of bridges. The project was also within the bottleneck reduction initiatives and thus was a perfect fit for the early 60-year design. The pavement was designed to be 10.5 inches thick with 15-ft joint spacing. The joint and load transfer design included stainless steel clad dowel bars and non-skewed joints with compression elastomeric pre-formed sealants. After scoping the project, a benefit cost analysis of possible alternatives was performed during the design recommendation process (Izevbekhai B.I. 2001). Project performed a life cycle cost analysis of the following 4 options:

Option 1: Do nothing but slight repairs and reconstruct later.

Option 2: Perform a White topping on the segment

Option 3: Construct a long life concrete pavement

Option 4: Construct a Bituminous pavement.

The LCCA led to the choice of a 60-year concrete design based on the results obtained and shown in

Table 2 and Figure 1 show the fiscal outlay and the Life cycle cost process for the 4 options.

Table 2: A 2001 Life Cycle Justification of a 60-Year Design Highway 100 Robbinsdale MN

	OPTION 1 Initial Minor Repairs			OPTION 2 Initial Whitetopping			OPTION 3 Long Life Concrete Const.			OPTION 4 Asphalt Pavement Const.		
Fix Year	FIX	FC× 10 ³	PW ×10 ³	FIX	FC× 10 ³	PW ×10 ³	FIX	FC× 10 ³	PW ×10 ³	FIX	FC× 10 ³	PW ×10 ³
1	Crack Repair & Micro	75	75	2002 White topping +	300	280. 0	Conc. Reconst	450	500	Bit. Reconst	450	450
11	Bit. Const.	500	324. 79	Crack Repair	66.5	43.2		0		Med. M & O'lay	300	195
18		0			0		Conc. Rep	150	74		0	
21	Mill & Overlay	250	109. 70	Conc. Reconst	500	219. 4		0		Med. M & O'lay	100	44
31	Thick M. & O'lay	250	74.1 1		0		Conc. Repair	150	46	Med. M & O'lay	100	30
38		0			0	32.0				Thick Mill & Overlay	200	90
39		0		Conc. Repairs	200	45.1					0	
41	Bit. Reconst	500	100. 14		0		Conc. Repair	150	35	Thick Mill & Overlay	250	50
46		0		Joint Repair	66.5	11.4		0			0	
51	Med .M. & O'lay	150	20.2 9	Conc. Repair	200	28.1		0		Med. M & O'lay	180	25
Sum			701. 16			635. 7		0	588			822
Residual Life		0	0		9	25.3		0	0		9	23
Sum Adjusted for Res. Life			701. 1			610. 3			588			799
			32.6 4			28.4 1			27.3 8			37.1 8

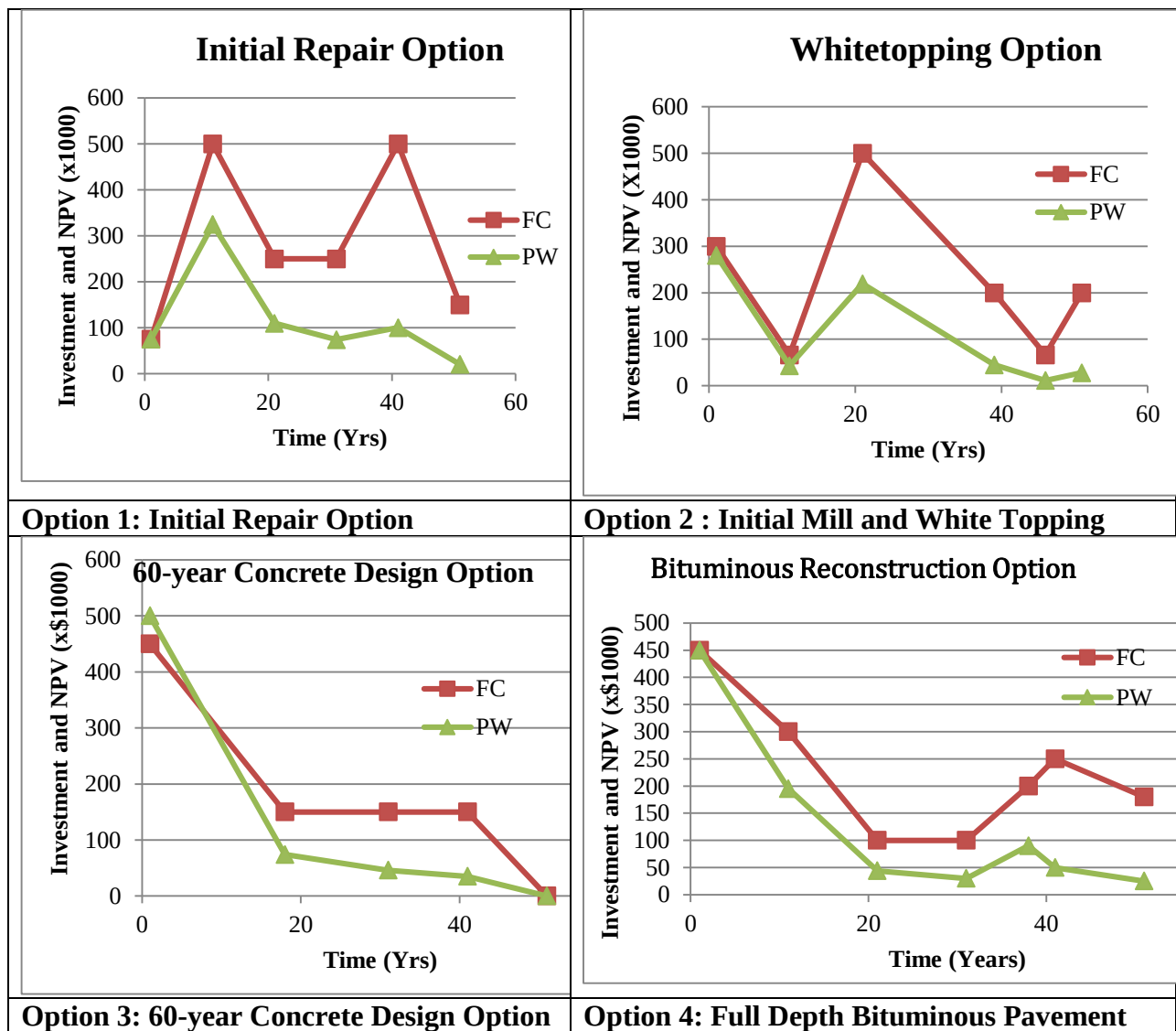


Figure 1: Investment and NPV Outlay for the Four Options

Incremental Benefit/Cost Analysis

The benefit computed is based on conveying the number of vehicles for 50 years. User-cost implicit in the curing time of concrete to achieve strength at opening to traffic were considered along with subsequent activities involve typical repair materials that are characterized by a higher rate of strength gain. Actual percentage of traffic flow was computed for other options by subtracting the percentage of time used for the predicted rehab activities. Other benefit implicit in reduced accidents and reduced delay were considered similar across the options. The benefit per hundred million vehicle miles were computed and prorated according to anticipated closure rates over the service period. Projected benefits are shown in Table 3. To ascertain cost-

effectiveness of options, planners must consider both the performance benefits, design feature and intervention maintenance/ repair costs when evaluating options. The performance curves were then combined with institutional thresholds to predict intervention activities. The current IRI in the segment is 80 inches per mile which is still at the performance stage for preventive maintenance. Investment and present value outlay are shown in Figure 2 and Table 1. Incremental benefit cost takes a more dynamic approach than a benefit / cost analysis as the name implies. It investigates the effect of moving from the least cost option through the next option on the ratio of the resulting incremental benefit to incremental cost. This is also called the “Challenger-Defender Method.” Alternatives were ordered from least expensive to most expensive. The analysis began by taking the first two alternatives when the least expensive alternative is called the “defender” and the more expensive alternative is called the “challenger.” The change in net benefits between these two alternatives is divided by the difference in net costs. If the result is greater than or equal to 1, the increase in benefit of the “challenging” alternative is equal to or greater than its increase in costs and the “challenger” then becomes the “defender” and is compared to the next alternative. If the result is less than 1.0, the current “defender” is retained and the new “challenger” becomes the next Alternative on the list. Comparisons of the challenger to defender are made until all Alternatives have been considered. The surviving defender is the most economically efficient

If more than one alternative is considered for a single project, an incremental benefit-cost ratio can be used to determine which alternative(s) are the most economically desirable (optimize additional benefits gained for the added cost). This can be represented by the following formula:

$$IB/C = \Delta PV (\text{Benefits}) / \Delta PV (\text{Costs})$$

(3)

IB/C = Incremental benefit-cost ratio

PV(x) = Change in the present value of x (between two Alternatives)

Where

I represents incremental (The Defender minus Challenger cost or benefit whichever is applicable)

B represents benefit and

C represents cost

Table 3: Process of Incremental B/C

Annual Benefit Alternative	Annual Benefit	Incremental Benefit (ΔB)	Annual Cost	Incremental Cost (ΔC)	Incremental B/C Ratio (ΔB/ΔC)
3	\$10,000.00		\$27.38		365.23
3 and 2		-\$1,000.00		\$1.03	-970.87
2	\$9,000.00		\$28.41		316.79
3 and 1		-\$2,000.00		\$5.26	-380.23
1	\$8,000.00		\$32.64		245.10

3 and 4		-\$3,000.00		\$9.80	-306.12
4	\$7,000.00		\$37.18		188.27

From this Table 2, Option 3 (lowest-cost Option) is first compared with Option 2, and Option 3 is preferred to Option 2 ($\Delta B/\Delta C = -970$). Option 2 is then excluded from further consideration. Option 3 is next compared with Option 1, and Option 1 (the higher cost Option) is not preferred, since $\Delta B/\Delta C = -380$ (less than 1.0). In this case, spending an additional \$1030 on Option 1 will yield \$5260 of additional benefits. Option B is then eliminated from further consideration. Finally, Options 1 and 4 are compared, and the additional \$3,240 in cost from Option 4 is compared with the \$9800 of additional benefits from Option 4. Thus, $\Delta B/\Delta C = -306$ between Options 3 and 4, which makes option 3 is the optimal solution based on incremental benefits and costs. Notice that Option 3 was selected in 2002 with a simple B/C ratio of 365 (Izevbekhai B.I. 2001). The result of a direct B/C analysis does not necessarily imply that the incremental B/C analysis would yield the same result. Congruity of both methods is not always guaranteed except the option is more strongly cost beneficial. From an economic lemma, the HPC appears economically viable and cost-effective as proven from the preceding analysis. However, since the 60-year design vision is still at the 15 year age, there must be a continuous re-evaluation for optimal timing of interventions within the 60 year cycle (Figure 3) as the performance curves are updated from real projects.

A WEIBULL ANALYSIS OF OPTIMAL TIME BETWEEN REPAIRS

The periods between construction, 1st 2nd 3rd and 4th repair in more than 200 test sections in the network were examined. These are indicated sequentially as MTBMA1 MTBMA2 MTBMA3 & MTBMA4 where MTBMA refers to mean time between maintenance activities. First background is laid of reliability analysis failure probability and survival probability.

Failure Probability, Survival Probability

In failure analysis, the time-to-failure information on can be presented in many forms that may be more useful for the study of causes of failure. For failure probability a graph can be drawn of the rise in the total fraction, $F(t)$, of the items that have failed by any given time t . If the sampled pavements are assumed to be representative the $F(t)$ is the fraction of all such pavements which can be expected to fail by the running time, t , since new. That is, for any given item, $F(t)$ = probability of failure before a running time t . Another expression of failure probability is survival probability: by contrast with the above fraction, $P(t)$, of items surviving at running time t could be tabulated and plotted clearly. $P(t) = 1 - F(t)$. We assume that all pavements studied were repaired only through a distress related condition process. The principal process for this evaluation, the Weibull distribution has the following parameters.

- The Scale Parameter (μ) which is the scale factor is proxy for the average time between maintenance
- Threshold Time to Failure (TTF) (t_0) is the age at which distresses begin to show up. It is a proxy for the threshold of preventive maintenance.

- The Shape parameter β which is called the Weibull slope, modulus or shape is a proxy for the failure mode: early mortality ($\beta < 1$); random/uniform mode ($\beta \cong 1$) or end of life or wear-out mode ($\beta > 1$). The goal is that (β should be $\gg 1$)

The probability density function of a Weibull random variable t is: [1]

$$f_t = \frac{\beta(t-t_0)^{\beta-1}}{\mu^\beta} e^{\left\{-\left[\frac{t-t_0}{\mu}\right]^\beta\right\}} \quad (4)$$

Where "t" is time and other parameters have been defined.

The concept of time to repair or mean time between maintenance actions (MTBMA) is the antithesis of failure rate as a higher rate of failure results in a lesser MTBMA. The Waloddi Weibull method facilitates the collection of various properties of failure in a singular equation where the value of the parameters indicates what type of distribution and (in consequence) what is the performance of the infrastructure. This can be extrapolated into the original life prediction of the 60-year design, considering the improvements made. Here the distributions examined were of concrete pavements of many years of performance prior to the introduction of long life concrete pavements. This choice to use the said data from the network was predicated on the premise that failure rate of the HPC would be less than that of the network and that results obtained from the regular concrete data will tend to be conservative when applied to HPC prediction. Author chose a purely analytic method of determining the Weibull parameters as these would not only describe the distribution but also serve as a learning-based criterion for determining optimal time to maintenance or rehabilitation of concrete pavements. The data for successive maintenance (MTBMA1 to MTBMA 4) were treated as separate distributions. An alternative approach would have been to track the entire service life in each project and treat the intervening maintenance as events. That approach was found to be unwieldy because of constantly disappearing d-records owing to factors extraneous to pavement condition. Limiting factor in the chosen analysis is the absence of a prediction of MTBMA_i from MTBMA (i-1) (i= 1, 2, 3, 4). Figure 2 exemplifies some features of the process from problem statement to interpretation of results.

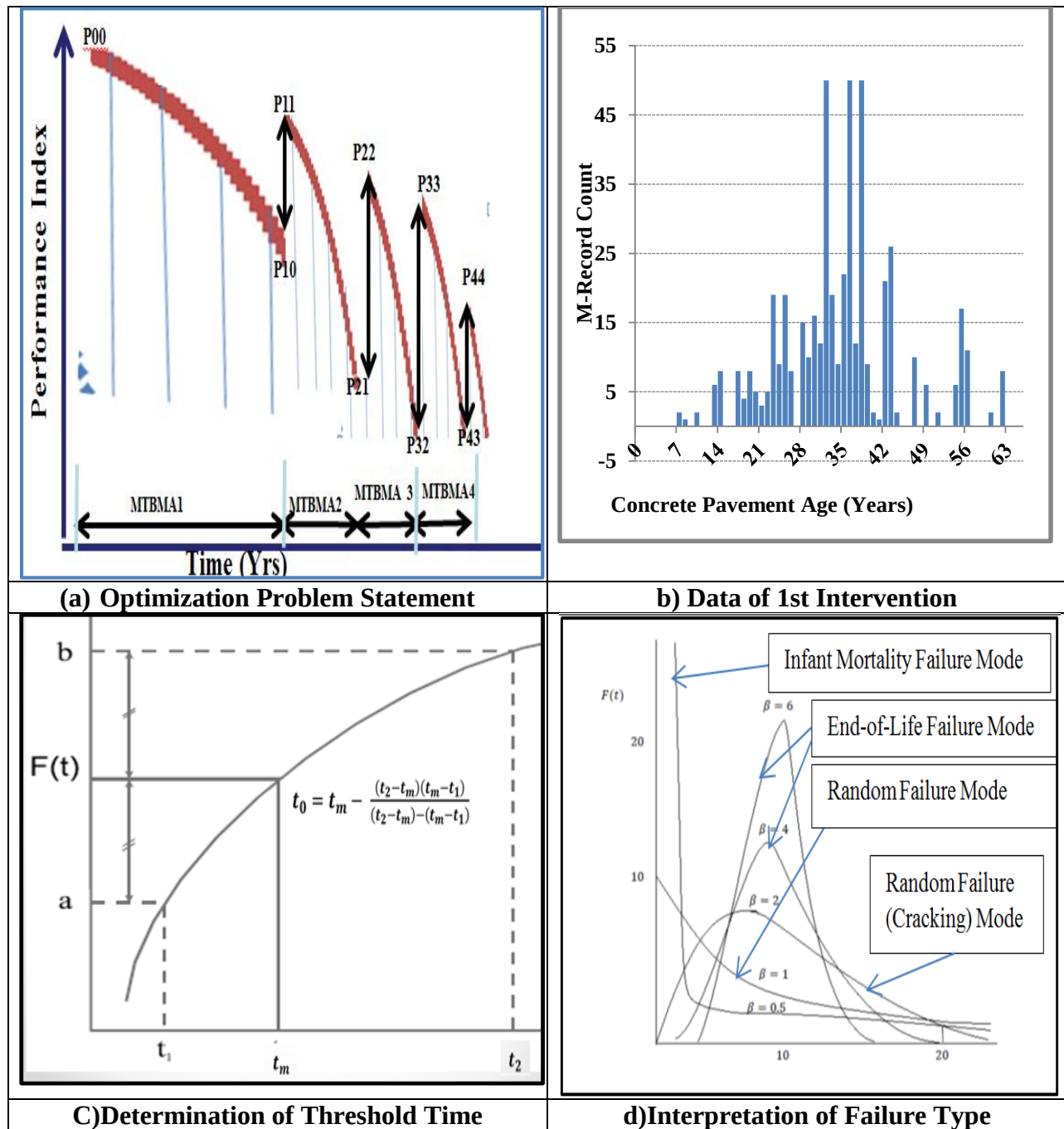


Figure 2: Problem Statement and Example of and Data Source, Data analysis and Data Interpretation

Steps for the Weibull analysis

Step 1: Data Assemblage:

Design records (or d-records) from over 200 projects in the network were assembled. For each d-record, the year of construction, year of interventions and the type of interventions were noted. For each project MTBMA 1 to 4 was noted. New projects had less MTBMA cycles yet. These allowed accumulation of MTBMA 1 to 4 separately and ready for analysis. Figure 2 shows the distribution of MTBMA1 for instance.

Step 2: Median Rank and Determination of Characteristic Life

For each MTBMA, a median rank percentile distribution was created that serves as F_t in the Weibull distribution. It allowed the percentile ranking of the time to maintenance activity to be ranked as percentile score. The cumulative frequency function was given by

$$F(t) = 1 - e^{\left\{-\left[\frac{t-t_0}{\mu}\right]^\beta\right\}} \quad (5)$$

Where t is time and t_0 is the reference time

$$\text{Failure Rate } (Z(t)) = (\beta/\mu) (t - t_0) \text{ (non-zero } t_0) \quad (6)$$

Note that since μ is the characteristic life defined as the interval between “ $t=0$ ” and the time at which it is expected that 63.2% of the items would have failed. When $t - t_0 = \mu$, $P(t) = \exp(-1) = 0.37$. The term t_0 refers to the threshold time to failure. The characteristic life is thus the 63.2nd percentile rank (Figure 3).

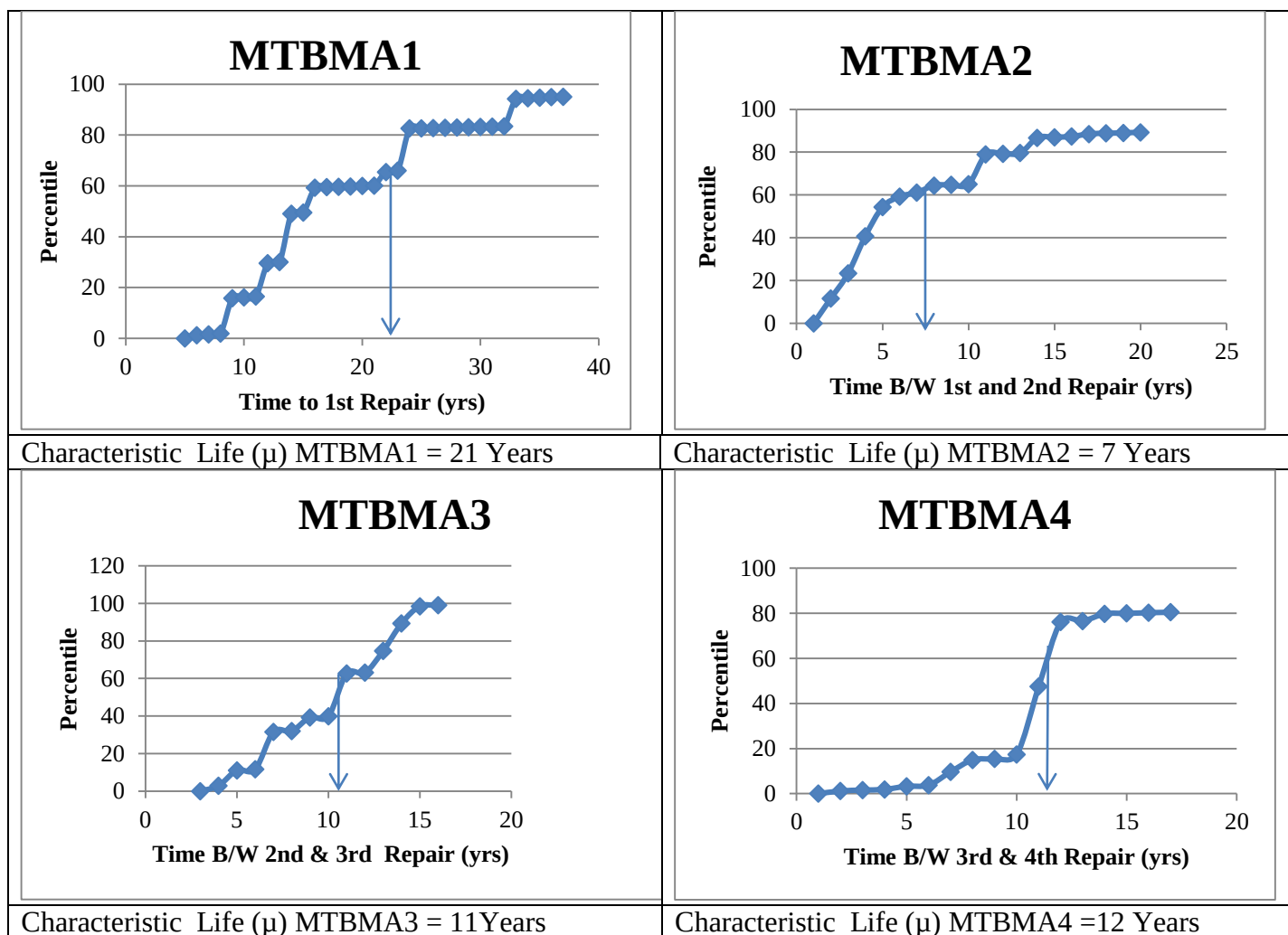


Figure 3: Ranked Percentile and Determination of Characteristic Life

Step 3: Determination of Threshold Time to Failure

The threshold time of failure is the infrastructure systems engineer's proxy for preventive maintenance threshold. It represents the time the system begins to experience distress. As a deterministic feature, it is determined from a reverse application of basic coordinate geometry (Figure 2) from which t_2 and t_1 require some skill, yet those 2 values are not stochastic. They correspond to the extreme abscissae of the longest concave portion of the percentile ranking. This is related to the parabolic assumption of the Simpsons rule for estimating areas. Here t_m is the median of the ordinates of t_1 and t_2 .

Step 5: Determination of Shape Function

In the weibull distribution various failure probability density functions are represented in a single expression. These are dependent on the shape factor β , that characterizes the failure mode.

The Cumulative probability function $F(t) = 1 - e^{\left\{-\left[\frac{t-t_0}{\mu}\right]^\beta\right\}}$
(6)

$$\text{From equation 5, } f(t) = \frac{\beta(t-t_0)^{\beta-1}}{\mu^\beta} e^{\left\{-\left[\frac{t-t_0}{\mu}\right]^\beta\right\}}$$

Taking Log twice, it can be shown that

$$\text{Log Log } (1 - F(t))^{-1} = \beta \log(t - t_0) + \beta \log \mu$$

(7)

Starting with MTBMA1 it was evident that if $\text{Log Log } (1 - F(t))^{-1}$ is plotted against $\log(t - t_0)$ a family of curves is generated with many hypothetical t_0 values. Using the t_0 value derived from the previous step the slope of the line is β . Threshold time to failure t_0 can also be detected by assuming certain realistic values for t_0 and examining which $t-t_0$ best represents a linear fit with $\text{Log Log } (1 - F(t))^{-1}$. Irrespective of the method, the true t_0 value will yield the right intercept $\beta \log \mu$ and also provide an alternative value of characteristic life. When t_0 , μ and β were determined, analysis was repeated for MTBMA (2 to 4). The process in figure 2c being a reliable method for determining t_0 was used.

Table 4: Summarized Result of Weibull Analysis

	MTBMA1	MTBMA2	MTBMA3	MTBMA4
MnDOT	17	10	10	10
Characteristic Life (μ)	21.5	7.5	10.5	11
Threshold TTF (to)	5	6	7	1
Shape Factor (β)	1.73	1.37	1.73	3.27
Performance Type	Random to Wear Out Log Normal Crack Development	Random	Random to Wear Out Log Normal Crack Development	Wear Out Spalling Punchout Faulting Rocking Panels

DISCUSSION OF IMPLICATION OF RESULTS OF WEIBULL ANALYSIS

The most important variable in the Weibull method is shape factor β because it tells the infrastructure systems engineer how the system is performing (early mortality, random failure or end-of-life failure mode).

Period Up to the First Repair (MTBMA1)

The characteristic Life, the threshold time to failure and the shape factors are shown in Table 3. This analysis shows threshold time (reasonable time for preventive maintenance) at the 5th year. Such preventive maintenance includes joint resealing. Such activities are not counted among the four treatments during the service life. The characteristic time to failure based on the 63.2 percentile time to failure or mean time before maintenance is 21 years. The MnDOT institutional value of 17 years is less than the Weibull value but steps have taken to revise it upward. The shape function indicates that the intervention characteristic is random to wear out but more indicative of wear-out since the factor $1.73 > 1$ (Figure 2d).

Period between the First Repair and the Second Repair (MTBMA2)

The characteristic life, the threshold time to failure and the shape factors are also shown in Table 3.

For the characteristic time to which the first signs of distresses occur is indicated by the characteristic time to failure. This analysis shows this phenomenon at the 6th year. It probably unrealistic to perform preventive maintenance. The characteristic time to failure based on the 63.2 percentile time to failure or mean time before maintenance is 7.5 years the difference between the characteristic time to failure and threshold time to failure is so short that preventive maintenance after first fix is unrealistic. The institutional value of 10 years is higher than this

Weibull value. The shape function indicates that the failure characteristic is random to wear out since the factor 1.3 is not significantly >1 (Figure 2d).

Period between the Second Repair and the Third Repair (MTBMA3)

The characteristic life, the threshold time to failure and the shape factors are also shown in Table 3.

The observed characteristic life and threshold time are approximately at the 7th year. It is probable therefore, that preventive maintenance. High t_0 may be attributed to the first major rehab that definitely includes diamond grinding that reduces faulting and significantly reduces dynamic loads. The characteristic time to failure based on the 63.2 percentile time to failure or mean time before maintenance is 10 years. The institutional value of 10 years is approximately equal to this Weibull value. The shape function indicates that the intervention characteristic which is assumed to be totally related to distress is random to wear-out but more towards wear-out since the factor 1.73 is significantly >1 in the Weibull scale. How this Weibull gradient is similar to that of the new pavement may also be related to diamond grinding. Shape functions close to 2 are attributed to non-life-threatening failure patterns such as concrete cracking and incipient warp and curl.

Period between the Third Repair and the Fourth Repair (MTBMA 4)

The characteristic life, the threshold time to failure and the shape factors are also shown in Table 3.

This analysis shows threshold time to failure at the 1st year which is indicative of old concrete. The characteristic life based on the 63.2 percentile time to failure or mean time before maintenance is 11 years. The institutional value of 10 years is approximately equal to this Weibull value. The shape function indicates that the intervention characteristic which is assumed to be totally related to distress is obviously non-random and evidently “wear-out” since the factor 3.24 is significantly >1 . Earliest deployment of the high performance concrete showed that the Interstate Highway 35W southbound section in Richfield Minnesota exhibited

General Observation

A review of one of the earliest Minnesota deployment of the high performance concrete showed that the segment is still at the stage of initial performance evidenced by an insignificant change in IRI. TH 35 HPC section southbound exhibited some seasonal fluctuation between 65 and 80 inches per mile (starting IRI) and northbound ranged between 110 and 120 inches per mile (starting IRI). The novelty of construction issues of the 60-year design in the northbound paved before the southbound may have reflected in this relative performance. In each direction, there is no obvious increase in IRI with time. The study concludes that the 60-year design is cost-effective but without prejudice to the need for continued monitoring and periodic reporting.

CONCLUSION AND RECOMMENDATION

This research paper examines the 60-year design philosophy, specifications and institutional maintenance and repair strategies. It examined the modifications and improvements made to the specification since 2000 when the first 60-year design project was designed and built. A typical high performance concrete project was reviewed. A life cycle cost and an incremental benefit cost analysis indicated that the high performance concrete option had the lowest NPV of investment and turned out to be the favorable option, as that scenario is proven analytically to be

a validation of the chosen option. it corroborated the 2001 choice of the high performance concrete pavement. A Weibull analysis of more than 200 concrete test sections in the network indicated that most of the sections exhibited "random / pseudo-random" failure modes in the first 3 rehabilitation / maintenance intervals but showed end of life mode ("wear-out") in the 4th rehabilitation / maintenance interval. Analysis showed that the characteristic life to first maintenance /rehabilitation was 21 years.

LIMITATIONS

By assumption, this analysis associated and attributed repair and rehabilitation work to distress condition response. A few projects may have been influenced by factors extraneous to pavement condition. On the conservative side, the concrete pavements examined were not designed for 60 years though some of them had outlived the 60 year period.

ACKNOWLEDGEMENTS

This paper was influenced by MnDOT Office of Materials and Road Research: Glenn Engstrom, Office Director, Curtis Turgeon, Pavement Manager, and Maria Masten, Concrete Engineer. Shelly Pedersen facilitated network data retrieval.

REFERENCES

- 1) Izevbekhai , B.I. and B.N Ubani (2015). Seven Year Performance Report on MnROAD High Performance Concrete Design Test Cell 53. MnDOT Research Report # 2015-38. URL:

<http://dotapp7.dot.state.mn.us/projectPages/pages/projectDetails.jsf?id=1053&type=CONTRACT>

Accessed 8/11/15

- 2) Burnham, T., B. Izevbekhai, and P. R. Rangaraju. (2006). "The Evolution of High-Performance Concrete Pavement Design in Minnesota," Proceedings, International Conference on Long-Life Concrete Pavements, pp. 135-151, held in Chicago, IL, October 25-27, 2006. Federal Highway Administration, Washington, DC.
- 3) Muench, S., L. Pierce, J. Uhlmeier, and K. Anderson. (2006). "The Evolution of Long-Life Concrete Pavements in Washington State," Proceedings: International Conference on Long-Life Concrete Pavements, pp. 19-35, held in Chicago, IL, October 25-27, 2006. Federal Highway Administration, Washington, DC.
- 4) Tayabji, S., and S. Lim (2006) Proceedings, International Conference on Long-Life Concrete Pavements, held in Chicago, IL, October 25-27, 2006, Federal Highway Administration, Washington, DC.
- 5) Izevbekhai B.I. Unpublished MnDOT Document 2001. MnDOT Design Recommendation Letter for project SP 2785-159. MnDOT Metro Division.