Performance of Class ¾ Inch Dense Graded HMA Pavements in Washington State

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Special Report

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15. SUPPLEMENTARY NOTES

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16. ABSTRACT

The performance of Class ¾ inch dense graded HMA pavements built between 1998 and 2002 were examined to determine their performing. No evidence was found that indicated that the Class ¾ inch mixes were performing any better or worse than other dense graded mixes built during the same time period.

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Introduction

The Washington State Department of Transportation has been designing and constructing dense graded hot mix asphalt (HMA) pavements according to the Superpave mix design process since 1997. The majority of the projects have utilized a Class ½ inch mix with less than 20 projects utilizing a Class ¾ inch mix up to the construction year 2003.

In 2004, the State Materials Laboratory was asked to investigate the premature failure of one of the Class ¾ inch mixes (Contract 5848, SR-395 East Elm Road to SR-17 Southbound) that was constructed in 2000. The forensic investigation determined a number of factors that lead to the premature failure (fatigue cracking in the wheel paths) on this roadway (Figure 1). These factors include:

- The mix design was conducted at a gyration level of 125, when the traffic volume and speed only required a 100 gyration mix. The gyration level alone was not the sole cause of failure on this project. However a mix designed with 125 gyrations will require lower asphalt contents than a 100 gyration mix. It is the resulting lower asphalt content that is believed to have contributed to the premature failure of this project.
- Presence of fine/clay particles. Unfortunately the project data had been purged prior to the initiation of the forensic study and the SE results which would indicate what proportion of the fines are clay were not available for this evaluation. The presence of the higher fine aggregate content during production would require higher asphalt contents than determined by the job mix formula (JMF).
- Although the average density on this project was 91.5 percent of maximum theoretical density (standard deviation of 1.10) and met WSDOT Standard Specifications (> 91 percent maximum theoretical density), it is lower than the average density that is typically measured, which for WSDOT projects is 93.3 percent of maximum theoretical density. A mix with lower asphalt content may require a higher compactive effort to obtain the same density level.

Figure 1. Premature fatigue cracking on SR-395

It was determined that the main cause of failure on this project was due to a "dry" mix, this impacted the ability to obtain sufficient compaction, resulting in higher permeability and making the pavement susceptible to aging, raveling and fatigue cracking. One of the recommendations of the forensic study was to conduct an analysis of all Class ¾ inch mixes used on the state highways to determine if similar performance issues exist. This paper will summarize information on the Class ¾ inch mixes constructed between 1998 and 2002 in Washington State according to density, binder type, and pavement performance (primarily cracking).

Literature Search

Nationally, since the inception of the Superpave mix design method in 1994, several states have reported permeability concerns associated with the use of coarse-graded mixes (mixes that are below the maximum density line at the No. 8 sieve). A critical element for adequate HMA performance is obtaining sufficient density and ultimately a mix that is impermeable to moisture.

A Florida Department of Transportation (FDOT) study (*1*) indicated that lift thickness impacts density and therefore permeability. The specific conclusions of the Florida study, as they relate to this paper, are:

- Presence of a distinct relationship between lift thickness and compactibility of coarse-graded mixes.
- Coarse-graded mixes require a higher density to reduce the permeability level to be equivalent to the fine-graded Marshall mixes, which equates to an in-place air void content of six to seven percent.
- As a results of this study, FDOT density requirements for coarse-graded mixes has been increased to a minimum of 94 percent of G_{mm}.
- FDOT minimum required lift thickness for coarse-graded mixes is 0.12 feet for the Class ⅜ inch mix, 0.15 feet for the Class ½ inch mix and 0.25 feet for the Class ¾ inch mix.

The National Center for Asphalt Technology (NCAT) has also conducted a number of studies (*2, 3, 4, 5*) to evaluate the relationship between in-place air voids, lift thickness and permeability. These studies have concluded that permeability, lift thickness and air voids are all interrelated. The studies summaries are as follows:

- Density, obtained under normal rolling conditions is clearly related to the ratio of thickness to the nominal maximum aggregate size (NMAS). For improved compactibility, fine-graded mixes should have a thickness to nominal maximum aggregate size (t/NMAS) ratio of greater than 3.0 and that for coarse-graded mixes should be greater than 4.0. Numbers less than these can be used, but generally more compactive effort would be required to obtain the desired density.
- To ensure that permeability is not a problem, in-place air voids should be between six and seven percent or lower, regardless of NMAS and grading.
- Even though significant scatter was noted within and between projects, most field results support that higher t/NMAS ratios generally provide lower void levels. Coarse-graded mixtures generally have higher permeability as compared to fine-graded mixtures at a given void level.

Research indicates that there is a relationship between aggregate size (coarse versus fine), lift thickness and compaction level. These factors, as they relate, to pavement performance will be the focus of the initial screening of the WSDOT Class ¾ inch mixes.

Lift Thickness Evaluation

With the Hveem mix design process, WSDOT typically placed Class A and B mixes (100-90 percent passing the ½ inch screen) at a lift thickness of 0.15 feet and Class E mixes (100-90 percent passing the 1.0 inch sieve) at a lift thickness of 0.20 feet. WSDOT noted that the Class E mix was more challenging to place due to aggregate segregation, but since this was primarily placed as a base course, concerns were minimized. Based solely on experience, WSDOT determined that the lift thickness used for the Hveem mix design procedure would be appropriate for the Superpave mix design process.

Table 1 illustrates the t/NMAS ratio for WSDOT Superpave mixes. All but one of the classes of mix (⅜ inch) meets the t/NMAS of 3.0. However, none of them meet the NCAT recommendations of a t/NMAS of 4.0 for coarse-graded mixes, though the Class ½ inch mix comes close (2).

Table 1. t/NMAS for WSDOT mixes.

Class of Mix	Lift Thickness (feet)	NMAS (inch)	t/NMAS	Minimum Lift Thickness Needed to Meet t/NMAS = 4 (feet)
⅜ inch	0.08	⅜	2.6	0.12
½ inch	0.15	½	3.6	0.16
¾ inch	0.20	¾	3.2	0.25
1.0 inch	0.25	1.0	3.0	0.33

Increasing the t/NMAS ratio to the NCAT recommendation of 4.0 would result in a relatively low financial impact for increasing just the Class ½ inch mix (which is the primary mix placed by WSDOT). However, WSDOT has not seen pavement failures related to the t/NMAS ratio and as a result there are no plans to change the lift thickness of the Class ½ inch HMA at this time.

There are many factors other than lift thickness that affect the performance of any pavement. A detailed investigation of a number of the Class ¾ inch mix projects will be undertaken to determine if there are specific factors that result in either superior or inferior performance.

Investigation of Class ¾ Inch HMA Performance

From 1998 to 2002, WSDOT designed and constructed 17 projects (considering nine of these projects are on divided highways, the number of different roadway sections that will be evaluated is 26) using the Class ¾ inch Superpave mix. The following will provide a comparison of the mix performance as it relates to in-place density, coarse versus fine-graded mixes, binder type and pavement performance (specifically, a measure of rutting, cracking and patching). The ability to directly relate mix properties (aggregate type, binder type, etc.) and mix production results (binder content, gradation and density) to pavement performance is very challenging due to variation in production and pavement performance; this is potentially only possible with research grade analysis where the mix, production and testing may be more tightly controlled. Therefore, this comparison is only intend to determine if there are any trends that can be noted with the performance of the Class ¾ inch mix as they relate to density, coarse or fine-grading and binder type.

Information concerning the state route number, contract title, milepost limits, direction, overlay depth, PG Grade, gyration level, gradation being fine or coarse and the year paved is summarized in Table 2. The gradation is considered coarse if the gradation curve falls predominately below the 45-power curve line and fine if it plots predominately above the line. Appendix A contains the gradation curves for the 17 projects included in the analysis. Eight of the projects were classified as coarse graded and nine as fine graded. Twenty-three of the 26 sections are located in the eastern portion of the state and three located west of the Cascades, all in the Northwest Region. Fifteen of the sections east of the Cascades are located in the South Central Region, five in the North Central Region and three in the Eastern Region.

A majority of the sections were built with a pavement thickness of 0.20 feet (21 sections), but there were two sections with a thickness of 0.15 feet, two with a thickness of 0.25 feet and one with paving lift thicknesses between 0.20 and 0.25 feet. There were two sections built in 1998, and six sections each in the other four years between 1999 and 2002. Fourteen of the sections were constructed using a PG 70-28 binder, seven with a PG 64-28, three with a PG 64-22 and one each with PG 58-34 and PG 64-34 binders.

State Route	Project Title	Cont. No.	Dir.	Begin MP	Ending MP	Overlay Depth	PG Grade	Gyration Level	Fine or Coarse	Year Paved
82	Valley Mall Blvd to Yakima R.	5373	I	36.31	38.86	0.20	70-28	109	Fine	1998
82	Valley Mall Blvd to Yakima R.	5373	D	36.31	38.86	0.20	70-28	109	Fine	1998
82	W. Prosser I/C to Oregon State Line	5581	I	82.14	90.17	0.20	70-28	100	Fine	1999
82	W. Prosser I/C to Oregon State Line	5581	D	84.35	90.15	0.20	70-28	100	Fine	1999
17	Lind Coulee Bridge to Vic. SR 90	5627	I	43.00	50.40	0.25	64-28	100	Coarse	1999
20	Narcisse Rd. to Vic. Spruce Canyon Rd.	5636	I	363.61	372.84	0.20	58-34	75	Coarse	1999
82	Naches R. Br. To Valley Mall Blvd.	5663	D	30.96	36.31	0.20	70-28	125	Coarse	1999
82	Naches R. Br. To Valley Mall Blvd.	5663	I	30.90	36.31	0.20	70-28	125	Coarse	1999
90	Vantage Br. To Burke	5779	I	137.67	148.44	0.20	64-28	75	Fine	2000
90	Vantage Br. To Burke	5779	D	137.67	148.50	0.20	64-28	75	Fine	2000
27	Fallon to Palouse	5803	I	8.78	14.54	0.20	64-28	75	Coarse	2000
395	E. Elm Rd. to SR 17	5848	D	36.10	45.36	0.20	70-28	125	Fine	2000
395	Kennewick Ave I/S to SR 182	5868	I	16.95	20.13	0.20	70-28	125	Coarse	2000
395	Kennewick Ave I/S to SR 182	5868	D	16.95	20.13	0.20	70-28	125	Coarse	2000
167	8th St. to 15th St. SW	5814	I	10.70	13.77	0.25	64-22	100	Coarse	2001
240	Stevens Dr. to SR 182	5977	I	30.64	34.60	0.20	64-28	100	Coarse	2001
240	Stevens Dr. to SR 182	5977	D	30.64	34.60	0.20	64-28	100	Coarse	2001
395	SR17 to Connell & SR260 to Adams Co. Line	6059	D	45.36	61.24	0.20	70-28	125	Fine	2001
90	Mercer Slough to 128th Ave SE	6104	D	9.72	10.55	0.15	64-22	100	Coarse	2001
90	Mercer Slough to 128th Ave SE	6104	I	9.72	10.55	0.15	64-22	100	Coarse	2001
20	Colville High School to Narcisse Cr.	6158	I	355.94	363.61	0.20	64-34	75	Coarse	2002
90	RR O'Xing to Adams Co. Line	6238	D	181.77	191.89	0.20	64-28	100	Fine	2002
90	RR O'Xing to Adams Co. Line	6238	I	181.77	191.89	0.20	64-28	100	Fine	2002
221	SR 14 to Prosser Hill	6308	I	0.03	23.01	0.20-0.25	64-28	100	Fine	2002
395	SR 82 to Kennewick Ave.	6369	I	13.10	16.87	0.20	70-28	125	Fine	2002
395	SR 82 to Kennewick Ave.	6369	D	13.10	16.87	0.20	70-28	125	Fine	2002

Table 2. Information on each section by year of construction.

Pavement Performance Model

As part of the Washington State Pavement Management System (WSPMS) (*6*), WSDOT annually collects (approximately 8,600 lane miles) pavement condition data according to the pavement structural condition (PSC), rutting and roughness. The PSC is a combined score that considers longitudinal cracking, transverse cracking, alligator cracking and patching. Pavement condition data has been collected almost every year since 1969 (total of over 25 years of data).

With years of pavement performance data, WSDOT is able to develop pavement performance curves that can be used to predict the timing for the next rehabilitation treatment. For example, Figure 2 illustrates a pavement that was constructed in 2000 and has five years of performance data (2001-2005). A non-linear regression curve is developed for each project based on its performance over time. This curve is reviewed and manually adjusted to mimic the curve that is normally produced for the particular type of pavement and location of the project. Using these curves WSDOT is able to predict the year in which a pavement reaches a specified distress level and therefore requires rehabilitation.

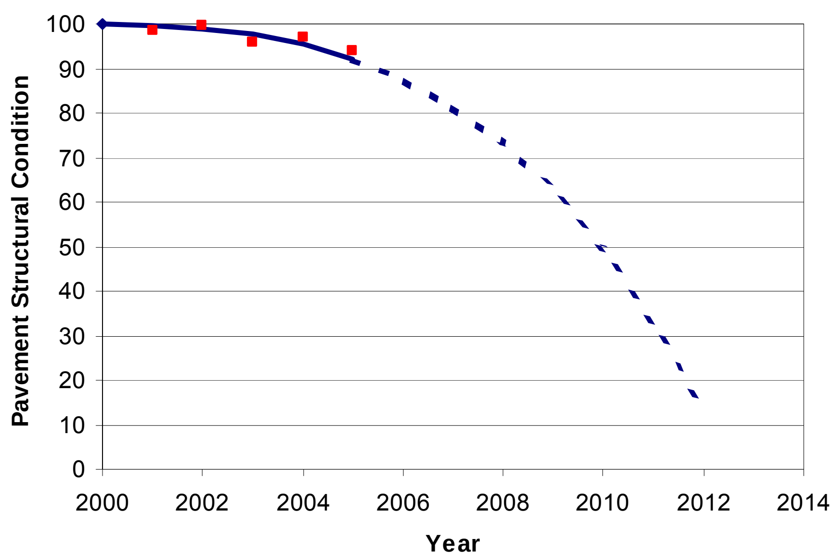

Figure 2. Example of WSDOT pavement performance prediction curve.

WSDOT has determined that a pavement can be maintained at its lowest life cycle cost if rehabilitation occurs when the pavement reaches a PSC score of 50. Therefore, the pavement characterized in Figure 2 will require rehabilitation in 2010 or at an age of 10 years.

Using the criteria of a PSC score of 50, a due year for required rehabilitation was determined for each section. Next the age at rehabilitation was calculated by subtracting the year paved from the due year. Finally, a change in life value was calculated as the difference between the age at rehabilitation and the average pavement life for each Region. The average pavement life for all classes of HMA for each Region is listed in Table 3.

Table 3. Average pavement life.

Region	Average Pavement Life
NW	17.2 years
SC	11.6 years
NC	10.1 years
O	15.9 years
E	12.0 years

The change in life value ranks the performance of a section in relationship to the current performance of other HMA pavements in a Region. A section that has a longer predicted life than the Region average has a positive change in life value, whereas a section with a predicted life less than the Region average has a negative value. Comparing the predicted life to the Region average life takes into account the environment and traffic in the area where the section is located. The sections are thus normalized for their location and can be compared on an even footing with other sections.

Table 4 lists the sections in rank order by their change in life value and Figure 3 shows graphically how each section ranked. Thirteen of the sections had positive values and 11 negative values. The positive values ranged from 0.4 years to 7.4 years of increased life. The sections on the negative side ranged from 0.2 years to 6.6 years of decreased life. Two projects were not ranked in the comparisons, SR-20, Narcisse Rd. to Vic. Spruce Canyon Rd., and SR-17, Fallon to Palouse, due to excessive flushing immediately after construction. The age at rehabilitation for these projects, as predicted by the WSPMS, is unrealistic because the WSPMS does not deduct points for flushing pavement. Both sections would have positive change in life values when, in fact, they should be considered failures from a performance standpoint.

Table 4. Section information ordered by Change in Life.

SR	Project Title	Cont. No.	Dir.	Milepost		PG Grade	Gyration Level	Year Paved	Due Year	Age at Rehab. (yr)	Region	Change in Life (yr)	AVE Density (pcf)	AC (%)
				Beg	End									
82	W. Prosser I/C to Oregon State Line	5581	D	84.35	90.15	70-28	100	1999	2018	19.0	SC	7.4	92.5	5.8
82	Valley Mall Blvd. to Yakima R.	5373	D	36.31	38.86	70-28	109	1998	2014	16.0	SC	4.4	93.6	5.2
90	Mercer Slough to 128th Ave SE	6104	I	9.72	10.55	64-22	100	2001	2021	20.0	NW	2.8	94.0	4.8
82	Valley Mall Blvd. to Yakima R.	5373	I	36.31	38.86	70-28	109	1998	2012	14.0	SC	2.4	93.6	5.2
82	Naches R. Br. To Valley Mall Blvd.	5663	D	30.96	36.31	70-28	125	1999	2013	14.0	SC	2.4	92.8	4.7
82	Naches R. Br. To Valley Mall Blvd.	5663	I	30.96	36.61	70-28	125	1999	2013	14.0	SC	2.4	92.8	4.7
395	Kennewick Ave I/S to SR 182	5868	D	16.95	20.13	70-28	125	2000	2014	14.0	SC	2.4	93.2	4.8
20	Colville High School to Narcisse Cr.	6158	I	355.94	363.61	64-34	75	2002	2016	14.0	E	2.0	93.6	4.6
82	W. Prosser I/C to Oregon State Line	5581	I	82.14	90.17	70-28	100	1999	2012	13.0	SC	1.4	92.5	5.8
240	Stevens Dr. to SR 182	5977	D	30.64	34.60	70-28	100	2001	2014	13.0	SC	1.4	93.9	5.0
395	SR17 to Connell & SR260 to Adams Co. Line	6059	D	45.36	61.24	70-28	125	2001	2014	13.0	SC	1.4	92.9	4.9
221	SR 14 to Prosser Hill	6308	I	0.03	23.01	64-28	100	2002	2015	13.0	SC	1.4	93.3	5.2
395	Kennewick Ave I/S to SR 182	5868	I	16.95	20.13	70-28	125	2000	2012	12.0	SC	0.4	93.2	4.8
167	8th St. to 15th St. SW	5814	I	10.70	13.77	64-22	100	2001	2018	17.0	NW	-0.2	93.2	5.0
240	Stevens Dr. to SR 182	5977	I	30.64	34.60	70-28	100	2001	2012	11.0	SC	-0.6	93.9	5.0
395	SR 82 to Kennewick Ave.	6369	D	13.10	16.87	70-28	125	2002	2013	11.0	SC	-0.6	92.6	4.4
90	Mercer Slough to 128th Ave SE	6104	D	9.72	10.55	64-22	100	2001	2017	16.0	NW	-1.2	94.0	4.8
395	SR 82 to Kennewick Ave.	6369	I	13.10	16.87	70-28	125	2002	2012	10.0	SC	-1.6	92.6	4.4
90	RR O'Xing to Adams Co. Line	6238	I	181.77	191.89	64-28	100	2002	2010	8.0	NC	-2.1	92.7	5.3
90	RR O'Xing to Adams Co. Line	6238	D	181.77	191.89	64-28	100	2002	2010	8.0	NC	-2.1	92.7	5.3
17	Lind Coulee Bridge to Vic SR 90	5627	I	43.00	50.40	64-28	100	1999	2007	8.0	NC	-2.1	93.0	4.6
90	Vantage Br. to Burke	5779	I	137.67	148.44	64-28	75	2000	2007	7.0	NC	-3.1	93.3	5.1
90	Vantage Br. to Burke	5779	D	137.67	148.50	64-28	75	2000	2007	7.0	NC	-3.1	93.3	5.1
395	E. Elm Rd. to SR 17	5848	D	36.10	45.36	70-28	125	2000	2005	5.0	SC	-6.6	92.0	5.1
20	Narcisse Rd. to Vic. Spruce Canyon Rd.	5636	I	363.61	372.84	58-34	75	1999	2013	14.0	E	-	93.9	5.2
27	Fallon to Palouse	5803	I	8.87	14.54	64-28	75	2000	2019	19.0	E	-	93.7	5.9

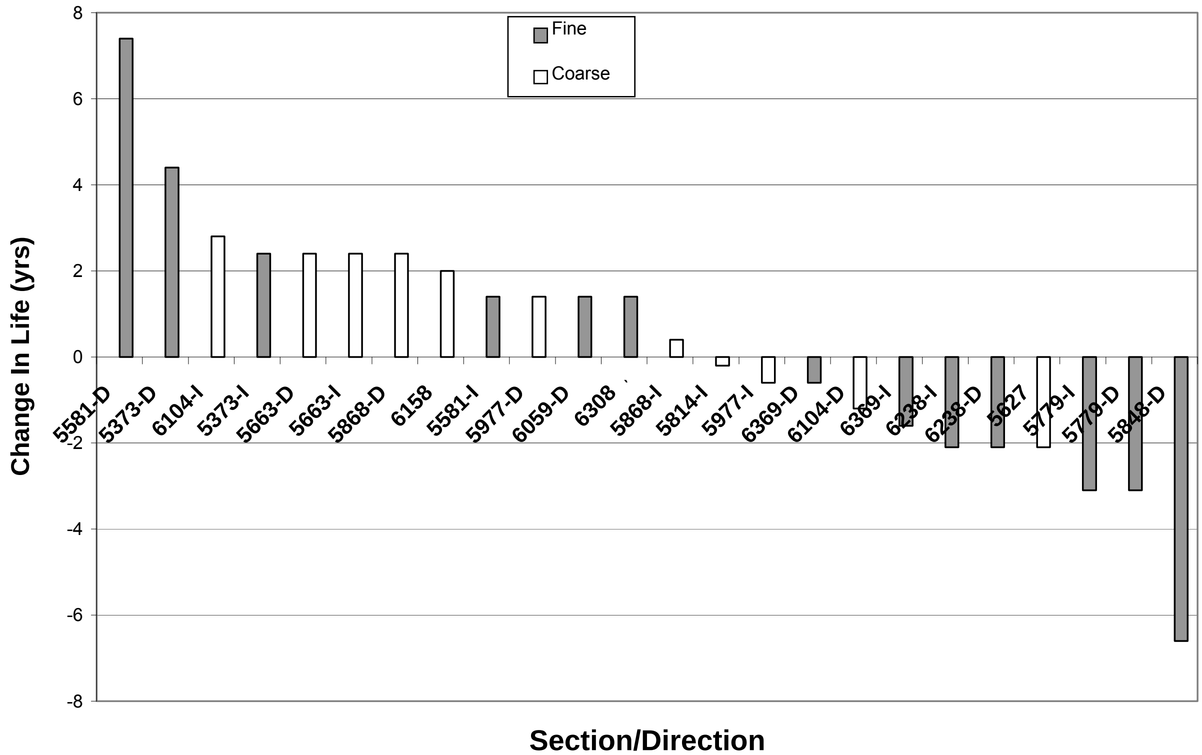

Figure 3. Change in life value for each section. Gray bars are mix design gradations on the fine side, white bars are gradations on the coarse side.

Analysis of Section Performance

A number of correlations were attempted to determine if one of a number of variables could be found that influenced the change in life value. The first item is the coarse and fine graded aspect of the mix designs. Figure 3 shows the random distribution of the fine and coarse mixes between the positive and negative changes in life values.

The second comparison was the average density of the pavement shown in Figure 4. There is an upward trend of increased life with higher average pavement density, but a correlation coefficient of 0.06 indicates very little correlation between density and predicted pavement life.

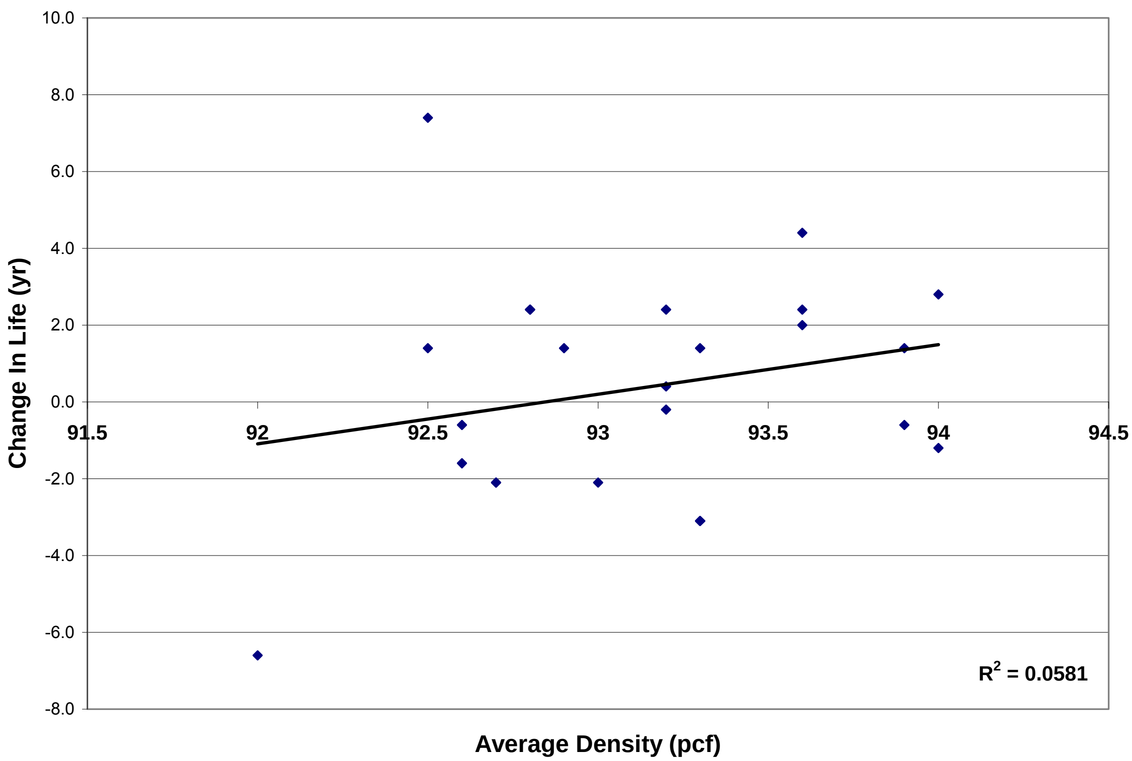

Figure 4. Change in life versus average density.

Figure 5 compares change in life to the design gyration level. The spread of change in life values for each gyration level would indicate that gyration level has no influence in pavement life.

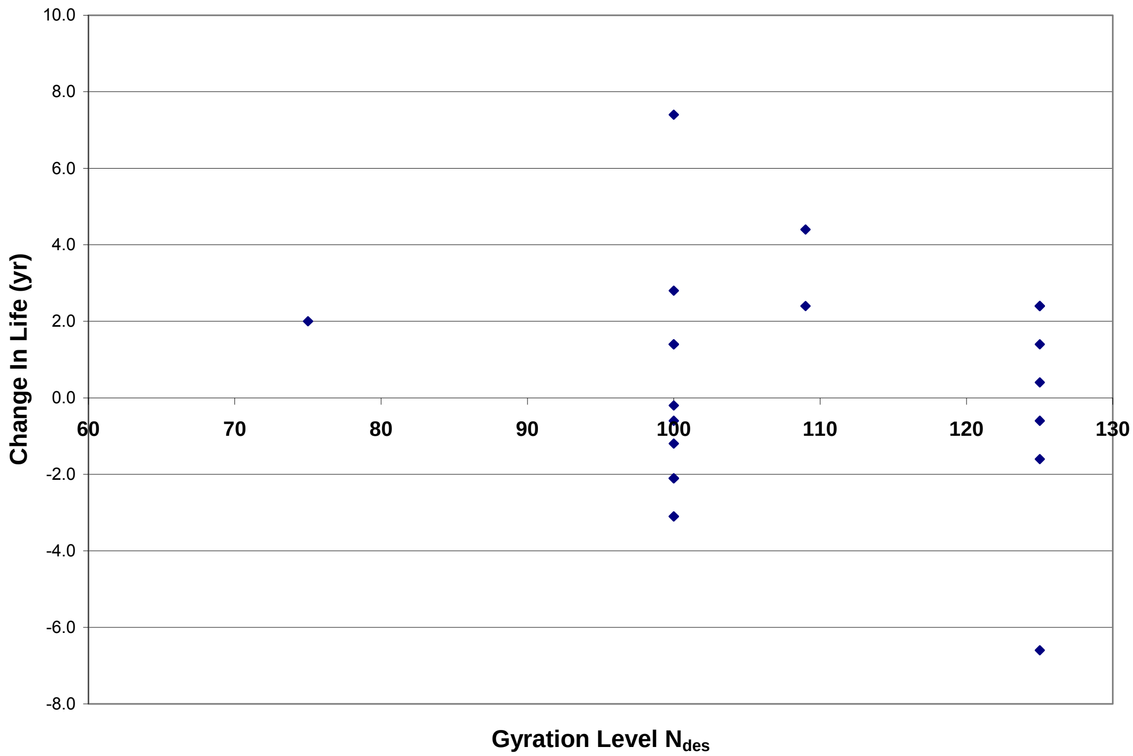

Figure 5. Change in life value versus N_{des} gyration level.

Figure 6 compares the change in life value to the binder type to examine if there are PG grades that are giving better performance. There is a weak indication that the PG 70-28 binder is providing longer life, but the correlation coefficient is extremely low. Additional data points would be needed to have confidence in this type of correlation.

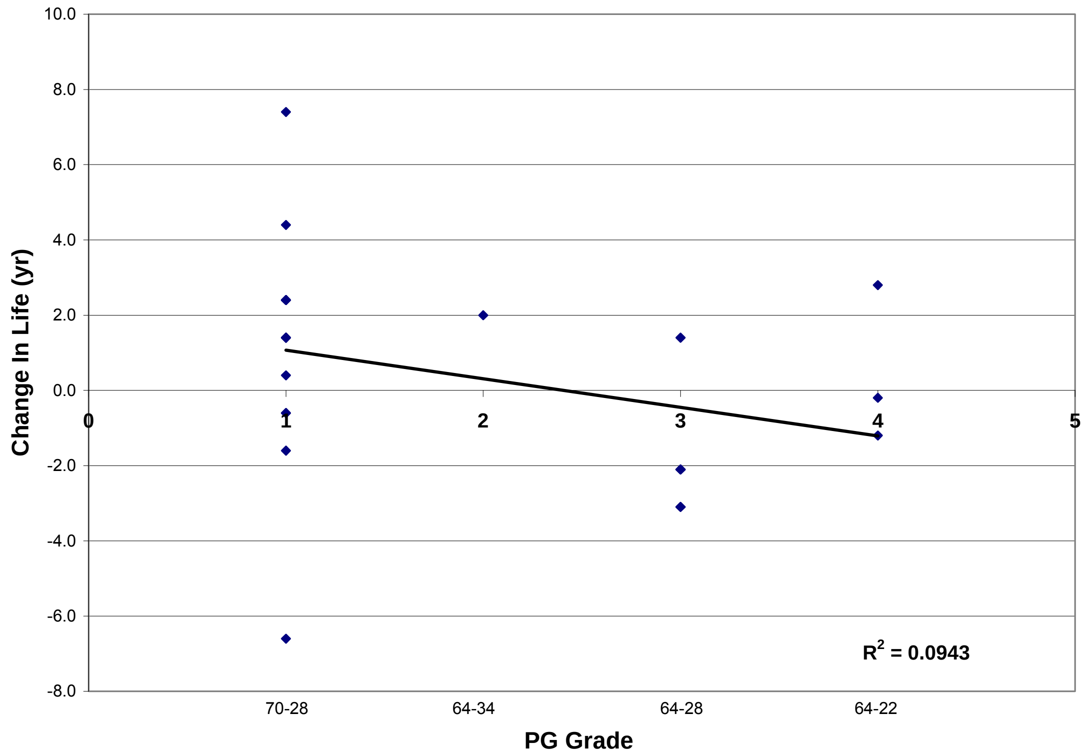

Figure 6. Change in life versus grade of binder.

Figure 7 shows the relationship between the change in life and the year of construction. It would not be expected that a relationship would exist between these two variables, however, the trend line indicates that the project built in 1998 and 1999 are performing somewhat better than those built in 2000, 2001, and 2002. The correlation coefficient, in fact, is the highest of any of the comparisons made for this set of data at 0.1234. This may suggest that more attention was paid to the projects built in earlier years when the Superpave design system was first implemented, however, with such a low correlation coefficient, that would be difficult to prove.

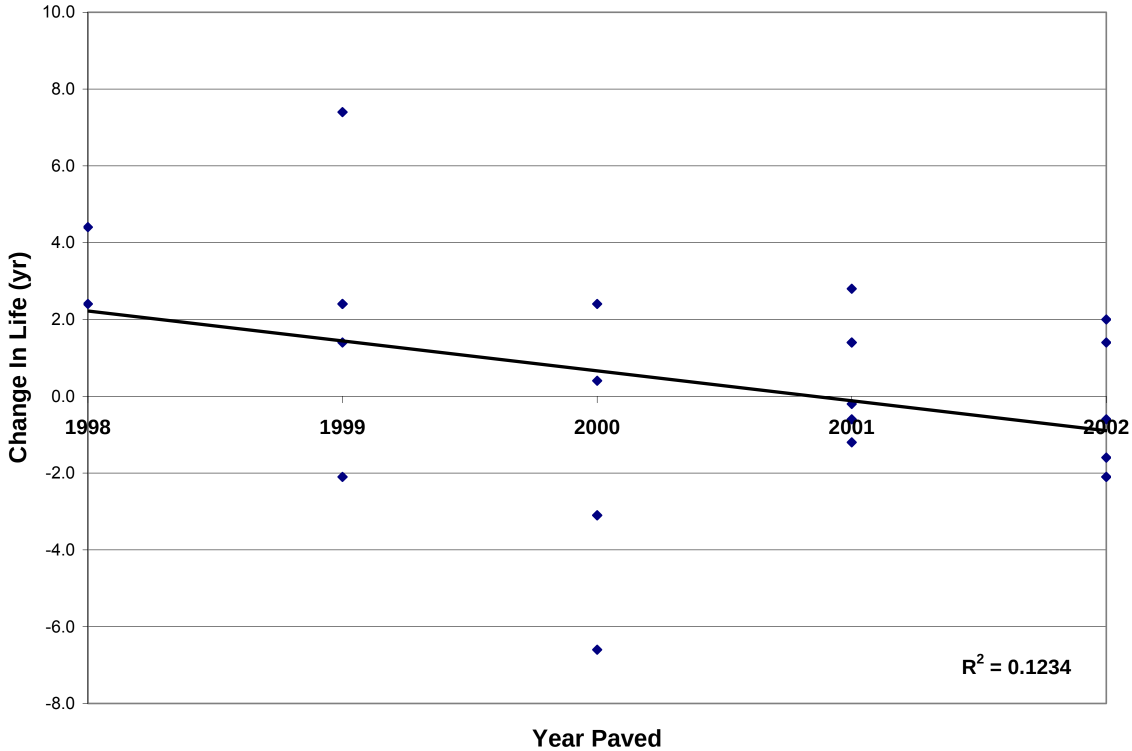

Figure 7. Change in Life versus year of construction.

Discussion of Results

The data presented in Table 4 indicates that 13 sections have positive changes in life and 13 have negative changes in life (counting the two flushing sections on the negative side). It might be concluded from this analysis that the choice to use a Class ¾ inch mix will result in success half of the time. A closer examination of the data gives a different picture. There are five sections that a change in life number of less than two years. It could be argued that the WSPMS equations are not accurate enough to predict the due year of a pavement within one year and possibly not even within two years. All five of the sections which fell less than two years behind their Region average pavement life were paved in either 2001 or 2002. Therefore, the equations that are predicting their life are based on four or five years of pavement condition data as compared with the sections built in 1998 and 1999 which have seven or eight years of data. The accuracy of the pavement life predictions would therefore be much lower for the sections built in 2001-2002 than those built in 1998 and 1999. It could be argued that these five sections could be assigned a change in life number of zero indicating that their expected life would be equal to the Region's average. Going one step further, there are three sections built in 2002 that have negative 2.1 changes in life. This also may not be an accurate prediction because there are only four points on the performance curve. Adding these three to the category with a change in life of zero would result in 21 projects out of 26 (81 percent) with positive changes in life and only five out of 26 (19 percent) on the negative side. This puts the performance of the Class ¾ inch mixes in a much more favorable light.

This does not imply that the Class ¾ inch mixes are not without their problems, as with any other class of mix there can be problems. There were three very definite failures in the 26 sections studies. Two sections, one on SR-20, Contract 5636, Narcisse Rd. to Vic. Spruce Canyon Rd. and one on SR-27, Contract 5803, Fallon to Palouse, flushed immediately after construction. A third section on SR-395, Contract 5848, E. Elm Rd. to SR-17 failed due to early cracking and raveling from insufficient binder in the mix, as noted earlier in this report. It is impossible to assign these failures to the fact that these were Class ¾ inch mixes since other sections using the same design criteria, same binder type and in similar climates and traffic conditions succeeded. For example, the good performing section on SR-20, C6158, Colville High School to Narcisse Cr., is immediately adjacent to the failing section on SR-20. The good performing section on SR-395, Contract 6059, SR-17 to Connell and SR-260 to Adams County Line, is immediately adjacent to the failing E. Elm Rd. to SR-17 section.

One problem with the Class ¾ inch mix pavements is visual appearance. The author made a visit to a number of the sections and described the appearance of the pavement. A common descriptor for many of the sections was that the surface texture was very coarse or open in appearance. Some sections were also described as dry or boney in appearance. These conditions appeared to be the result of the loss of the fine aggregate portion of the mix from the surface, possibly as a result of studded tire action. In many cases, the areas outside of the wheel paths or in the center of the lane did not appear to have the coarse texture and dry appearance. Photos of many of the sections are included in Appendix B. It should be noted that it is very difficult to capture the texture of a pavement in a photograph.

Conclusions

The attributes of each section such as the fine or coarseness of the aggregate gradation, the average density, the gyration level, the binder type and the year paved were compared with the change in life value to determine if the Class ¾ inch mixes in Washington State are performing any better or worse than other classes of mixes. None of the comparisons provided conclusive evidence of any correlation between performance and these variables. An examination of the change in life value indicated that 21 of the 26 projects were performing equal to or better than the average pavement life for the Region in which they were located. Based on this evidence it was concluded that the Class ¾ inch mixes where not performing any better or worse than other classes of mixes.

References

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2 Brown, E. Ray (2004). Relationship of Air Voids, Lift Thickness, and Permeability in Hot Mix Asphalt Pavements**,** *NCHRP Report 531*, Transportation Research Board.

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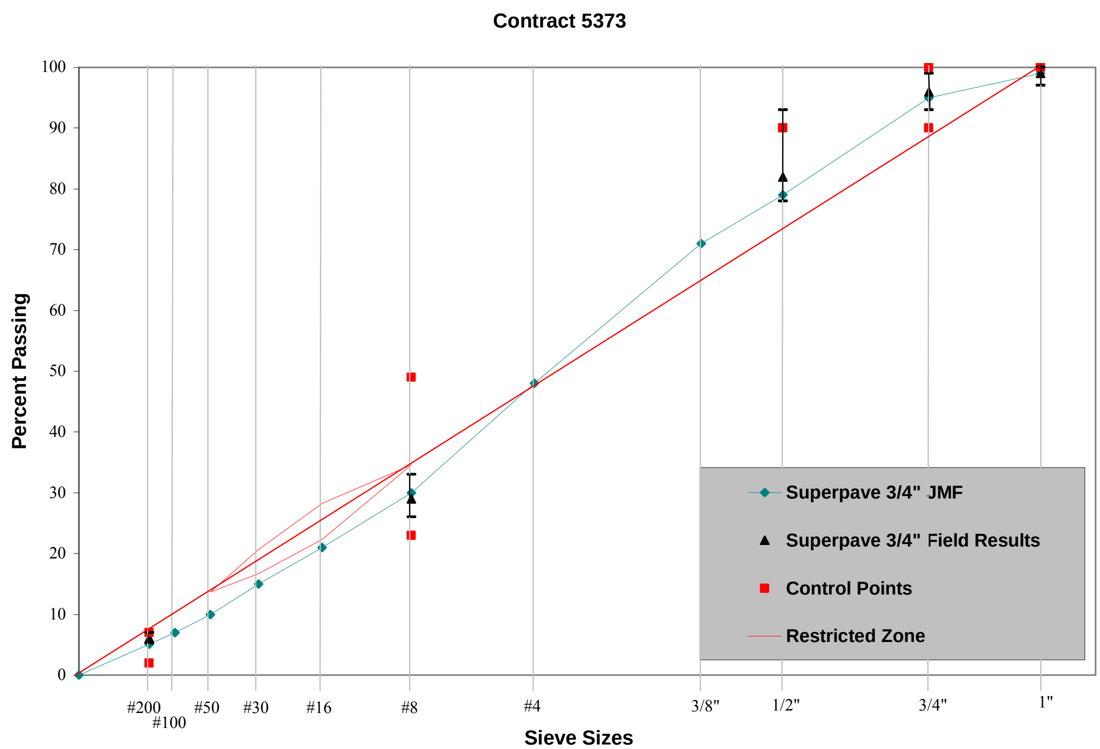
4 Cooley, L.A., Prowell, B.D., Brown, E.R. (2002). Issues Pertaining to the Permeability Characteristics of Coarse Graded Superpave Mixes, *NCAT Report 02-06*, http://ntl.bts.gov/lib/24000/24600/24625/rep02-06.pdf.

5 Cooley, L.A. (1999). Permeability of Superpave Mixtures: Evaluation of Field Permeameters, NCAT Report No. 99-1, http://www.eng.auburn.edu/center/ncat/reports/rep99-1.pdf.

6 Washington State Department of Transportation (2005). Pavement Management System, Olympia, Washington.

Appendix A

Gradation Curves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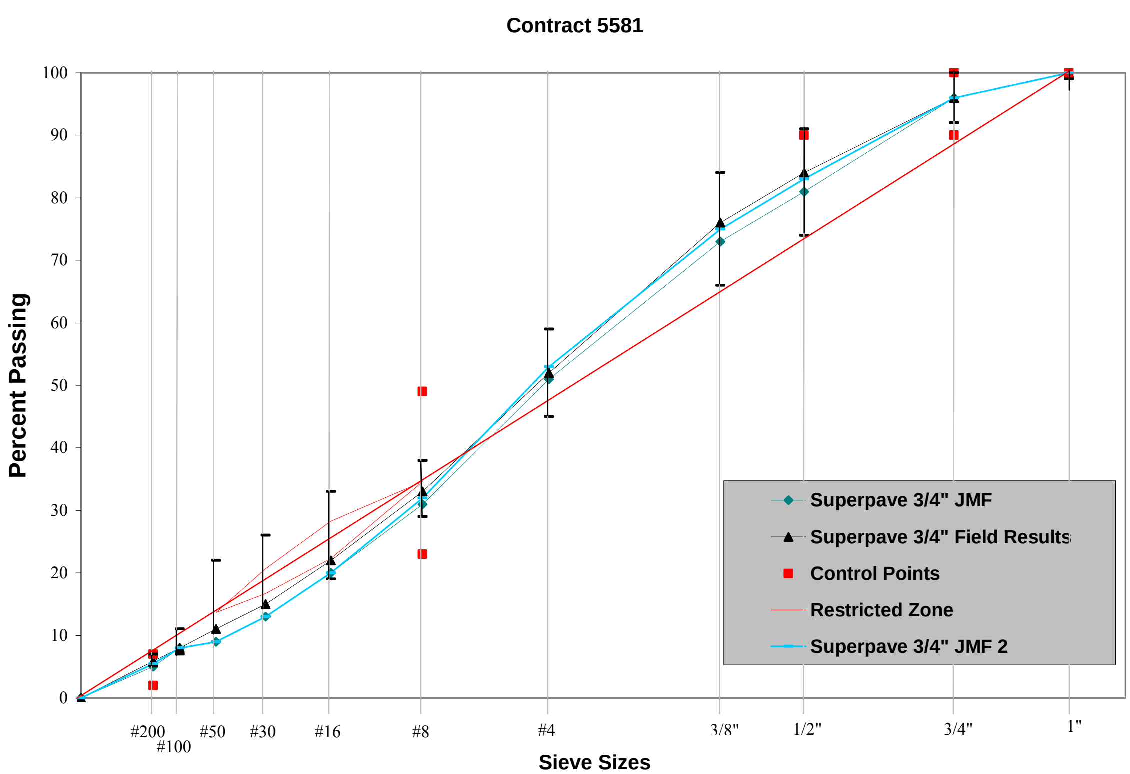
Superpave 3/4" Field Results
Control Points
Restricted Zone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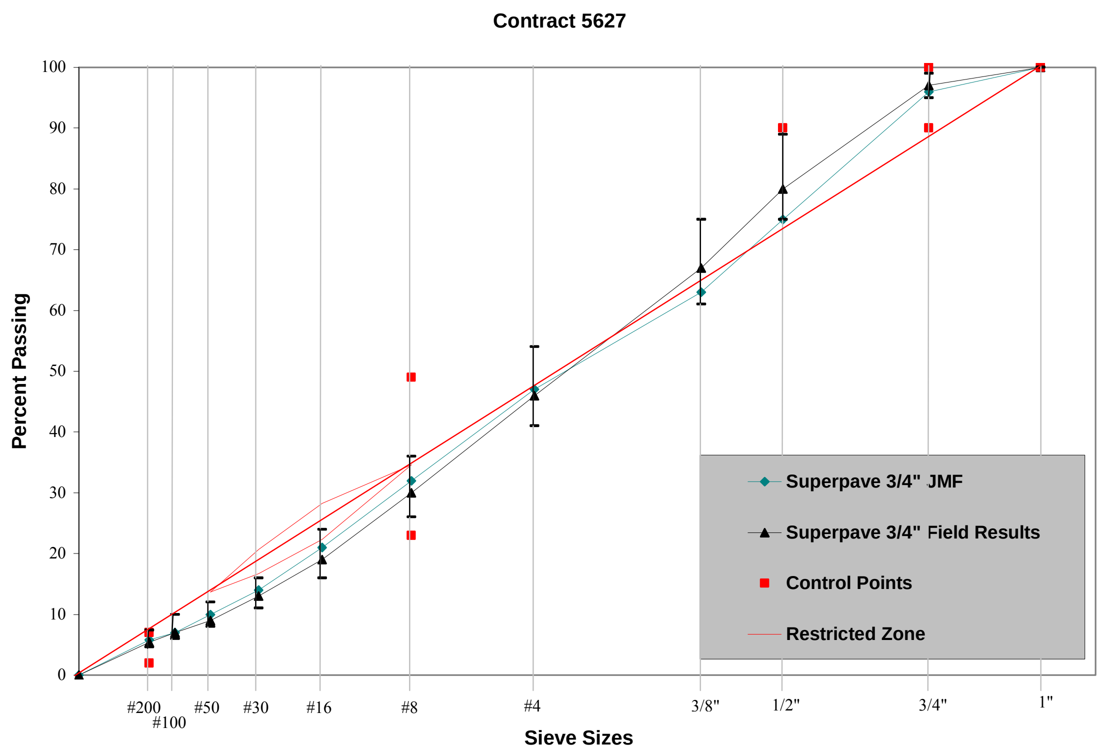
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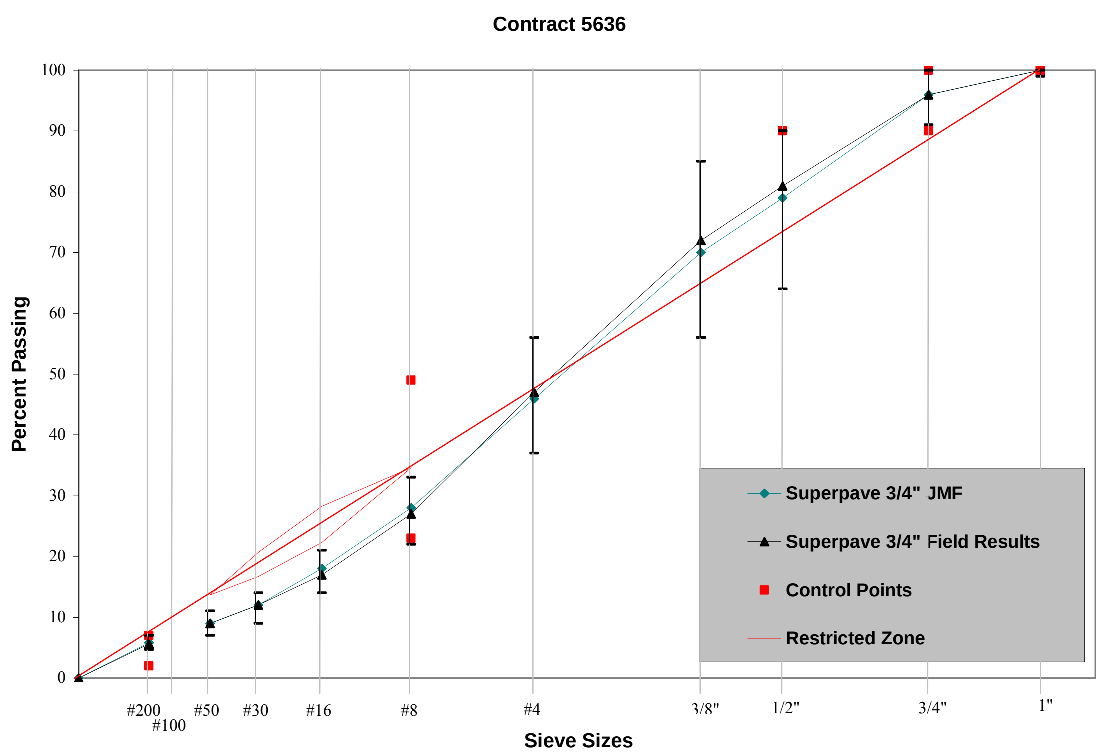
Superpave 3/4" Field Results
Control Points
Restricted Zone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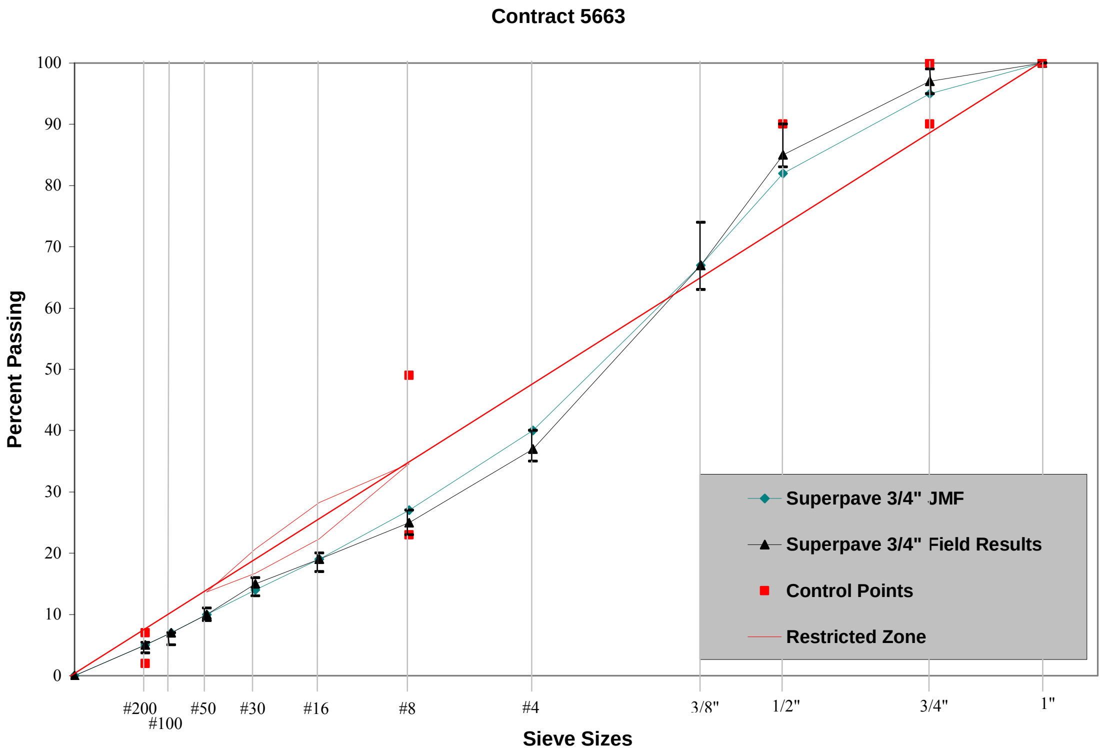
Control Points
Restricted Zone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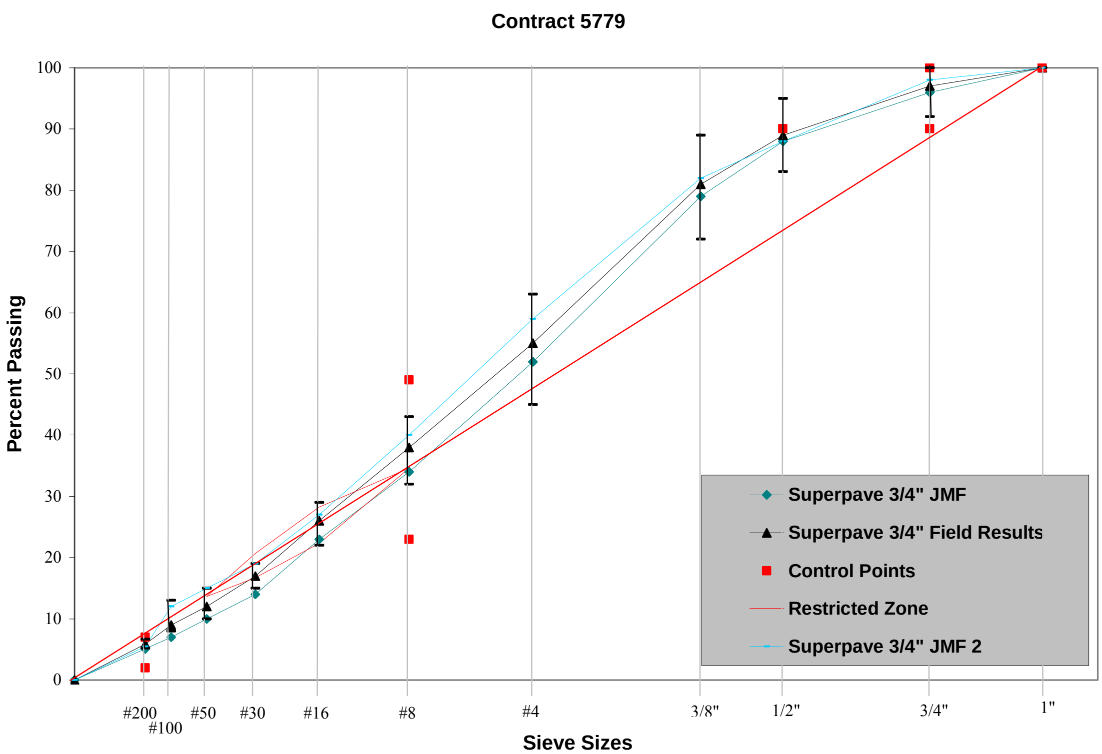
Control Points
Restricted Zone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Control Points
Restricted Zone
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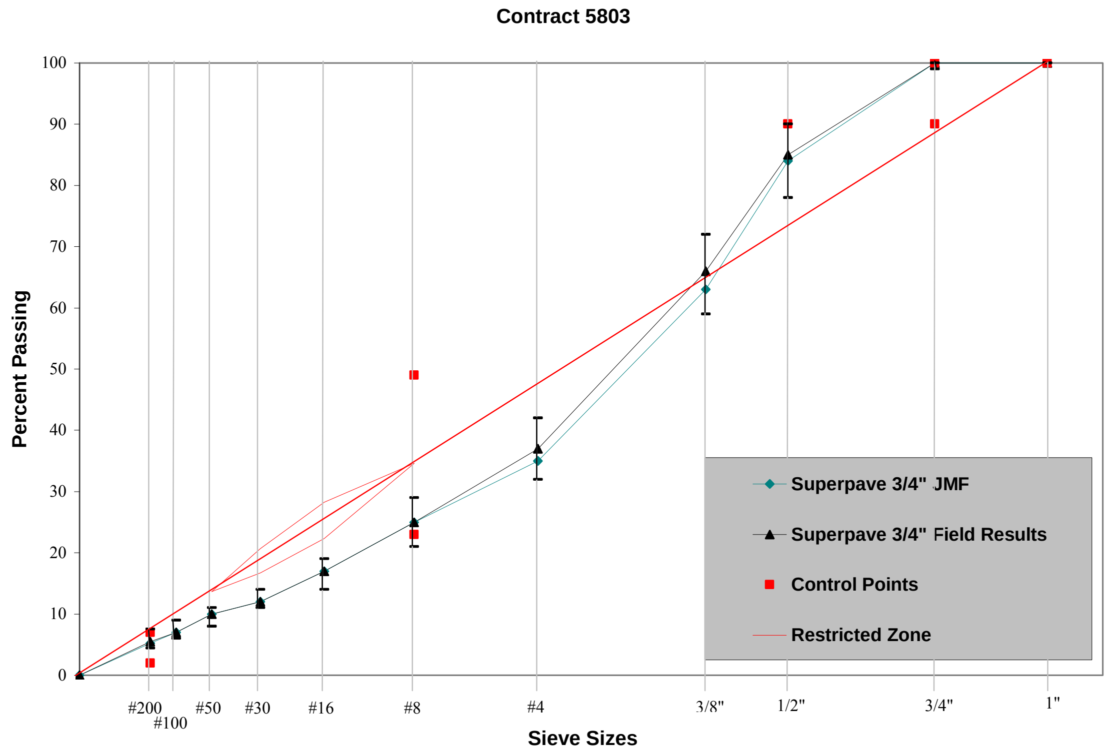
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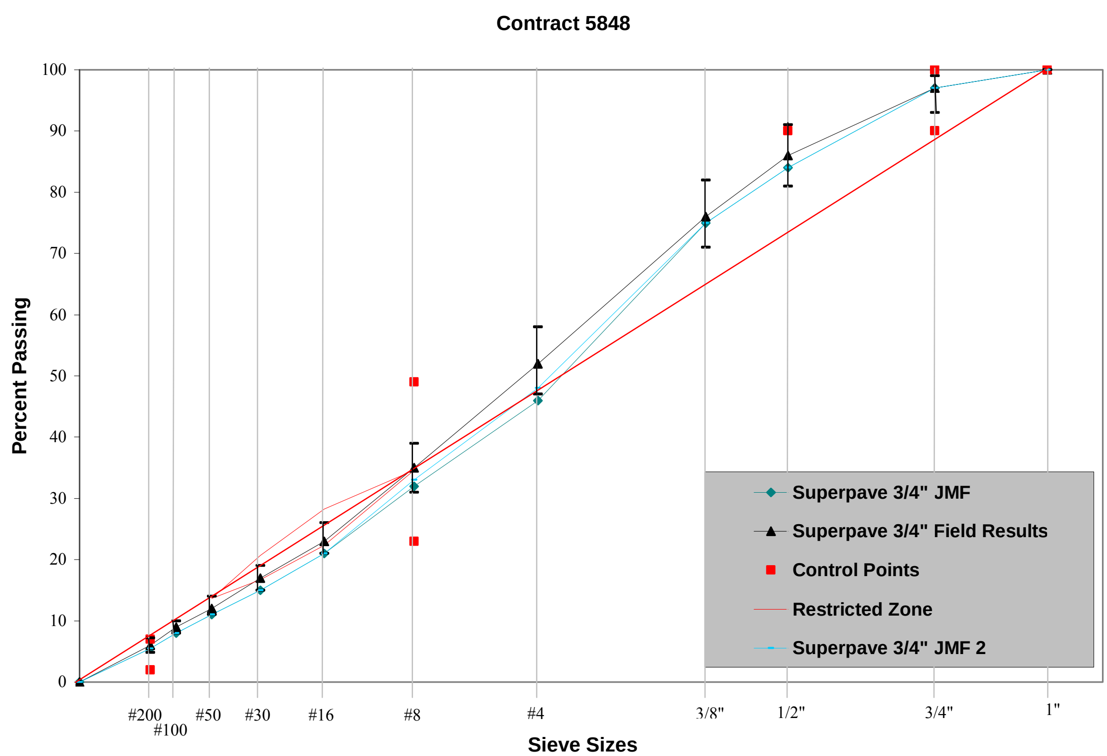
Control Points
Restricted Zone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Control Points
Restricted Zone
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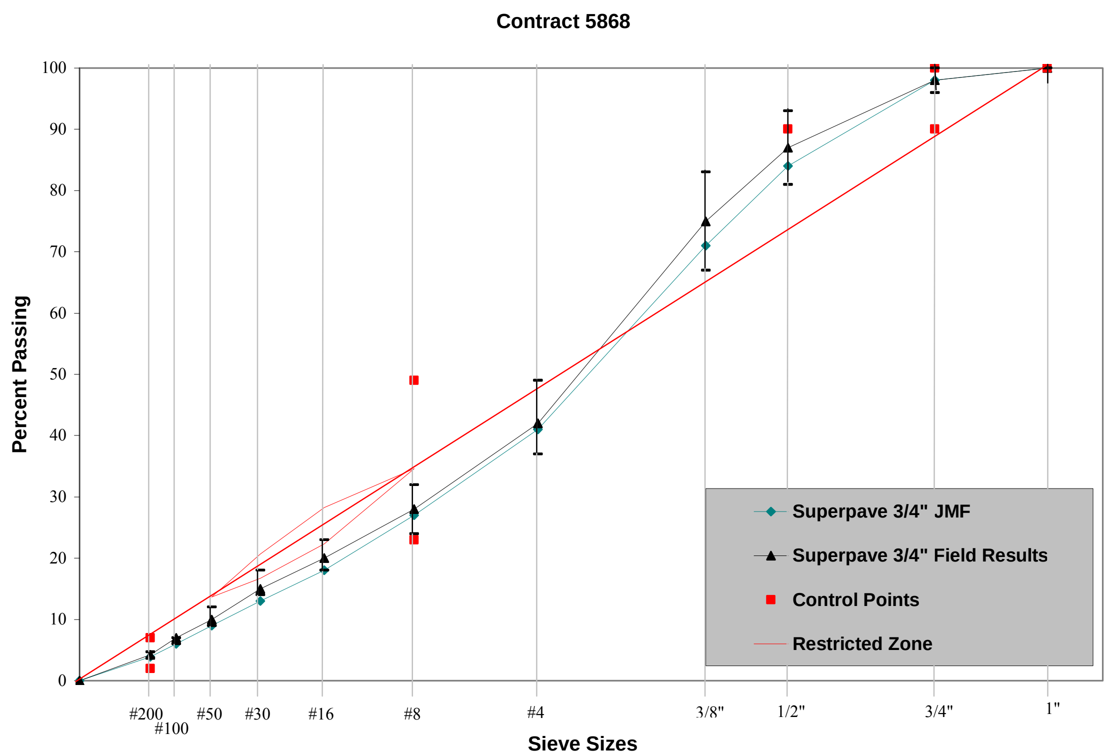
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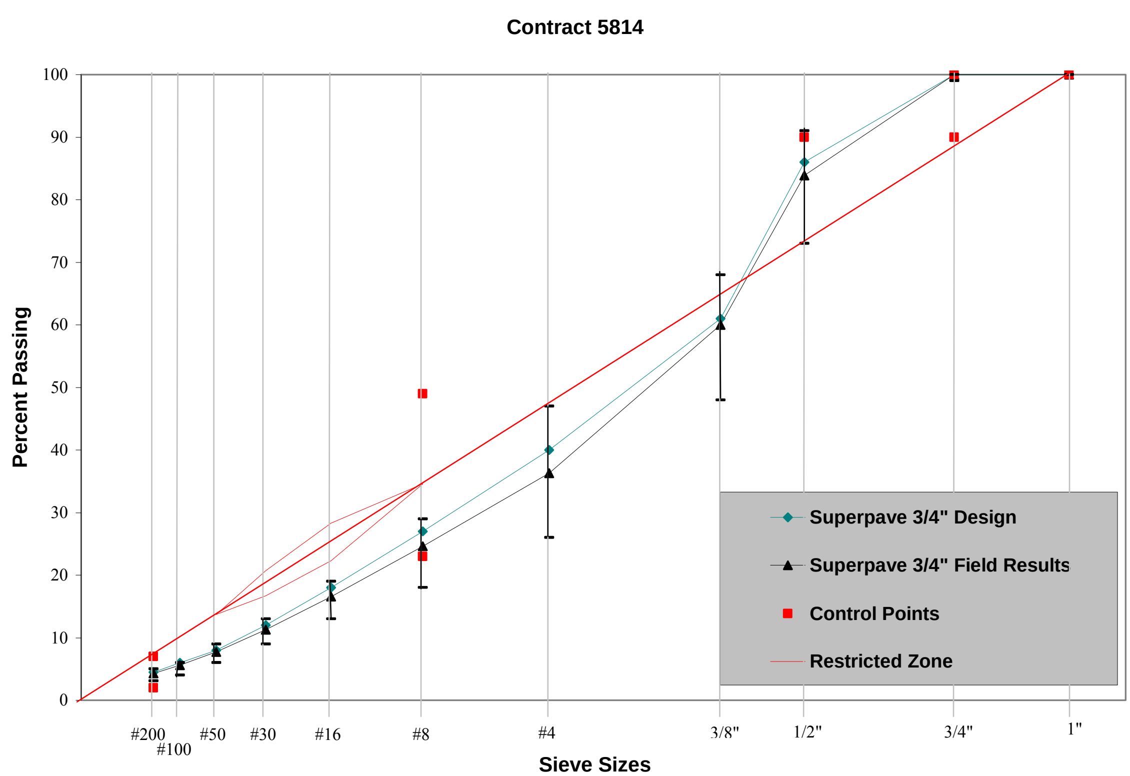
Sieve Sizes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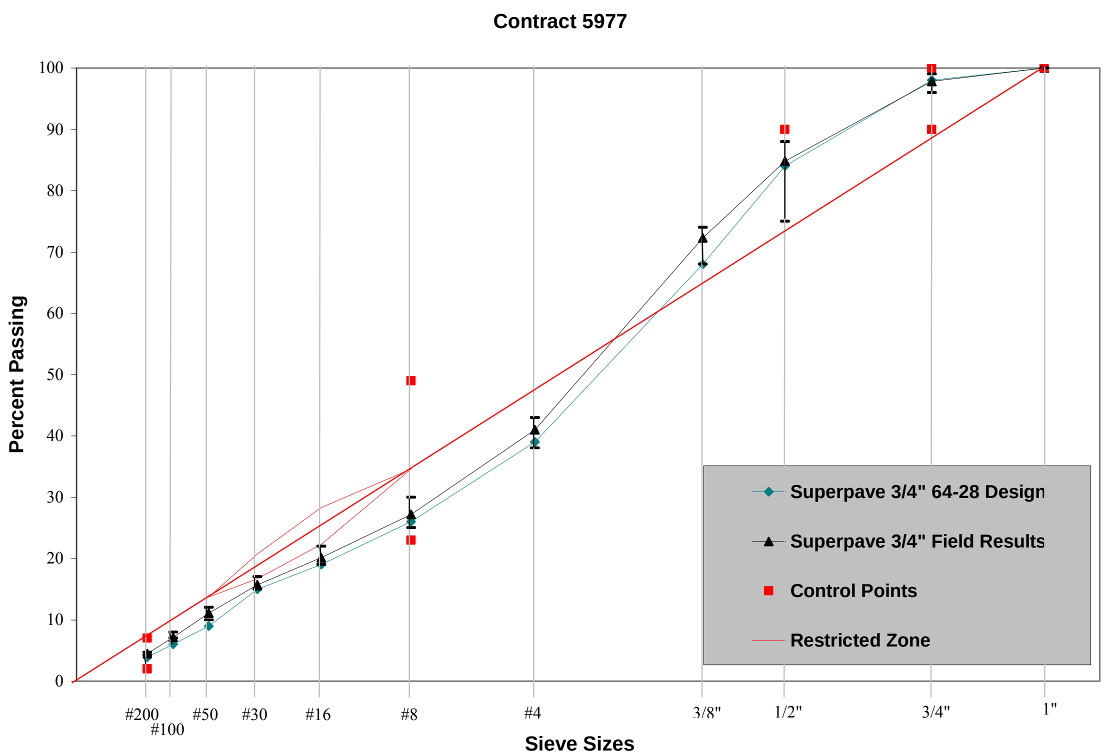
Sieve Sizes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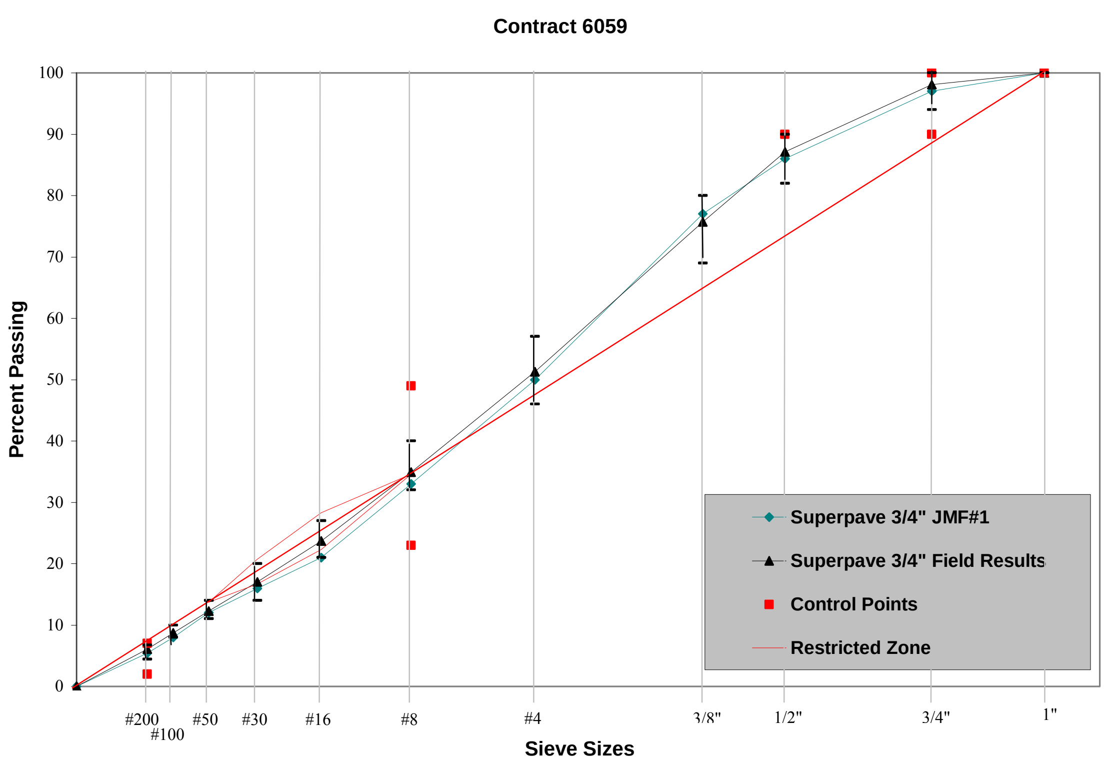
Restricted Zone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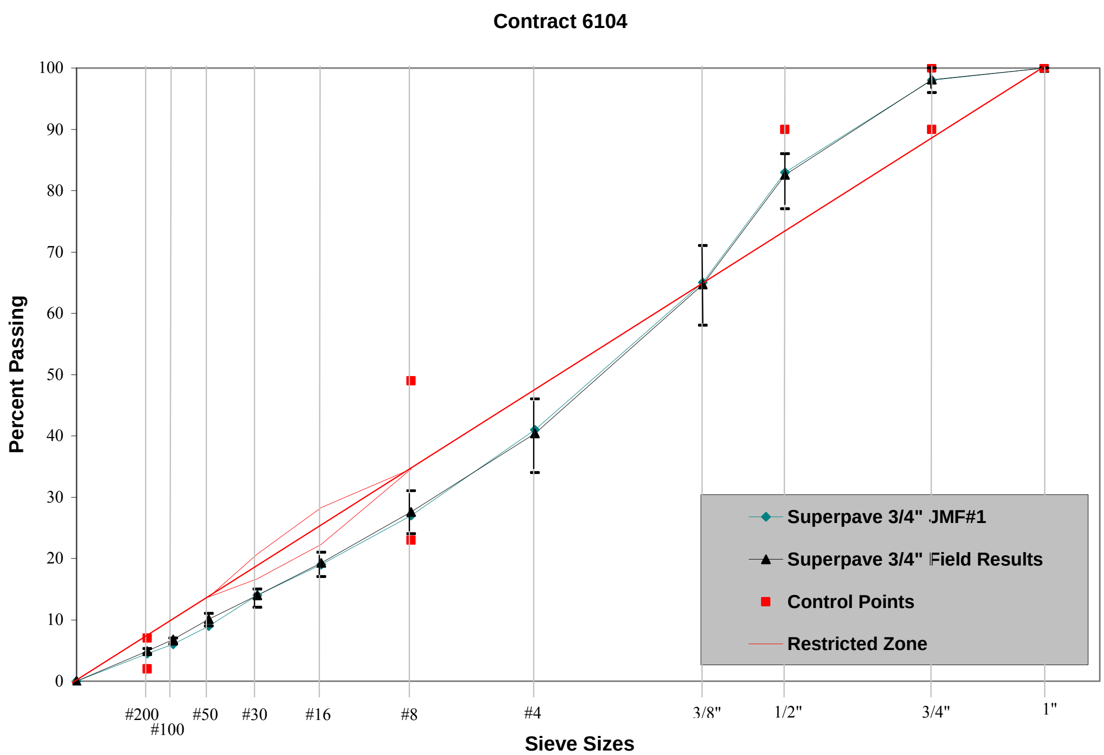
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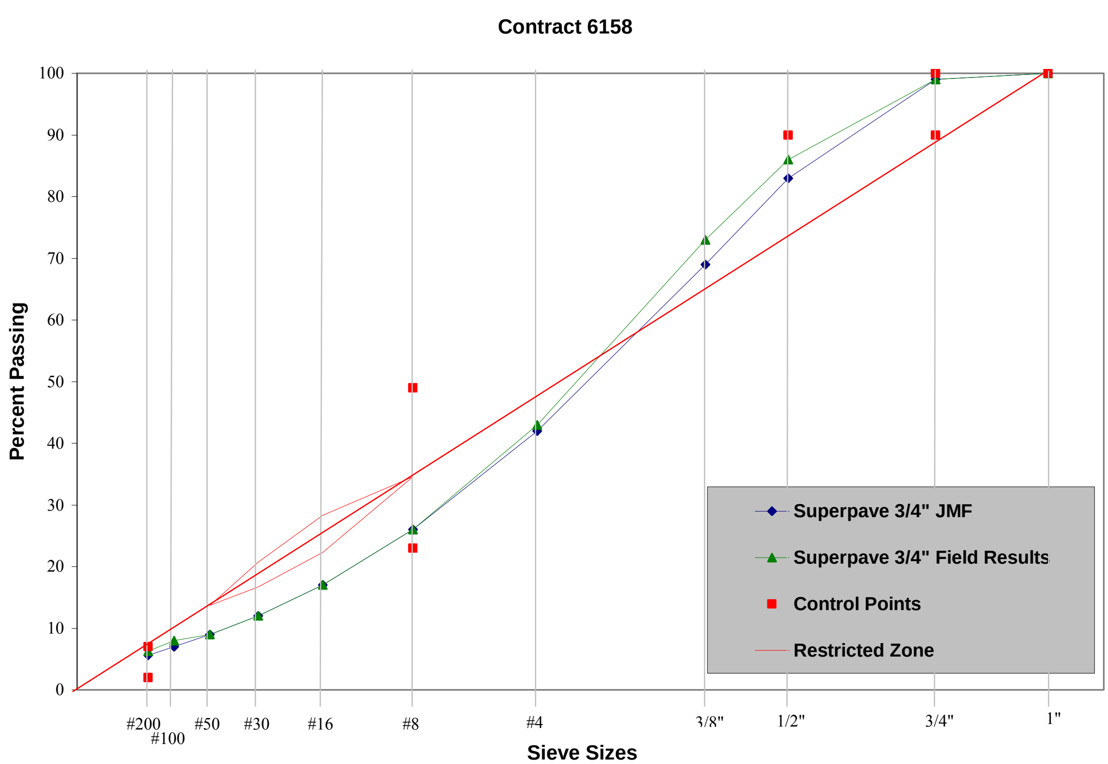
Control Points
Restricted Zone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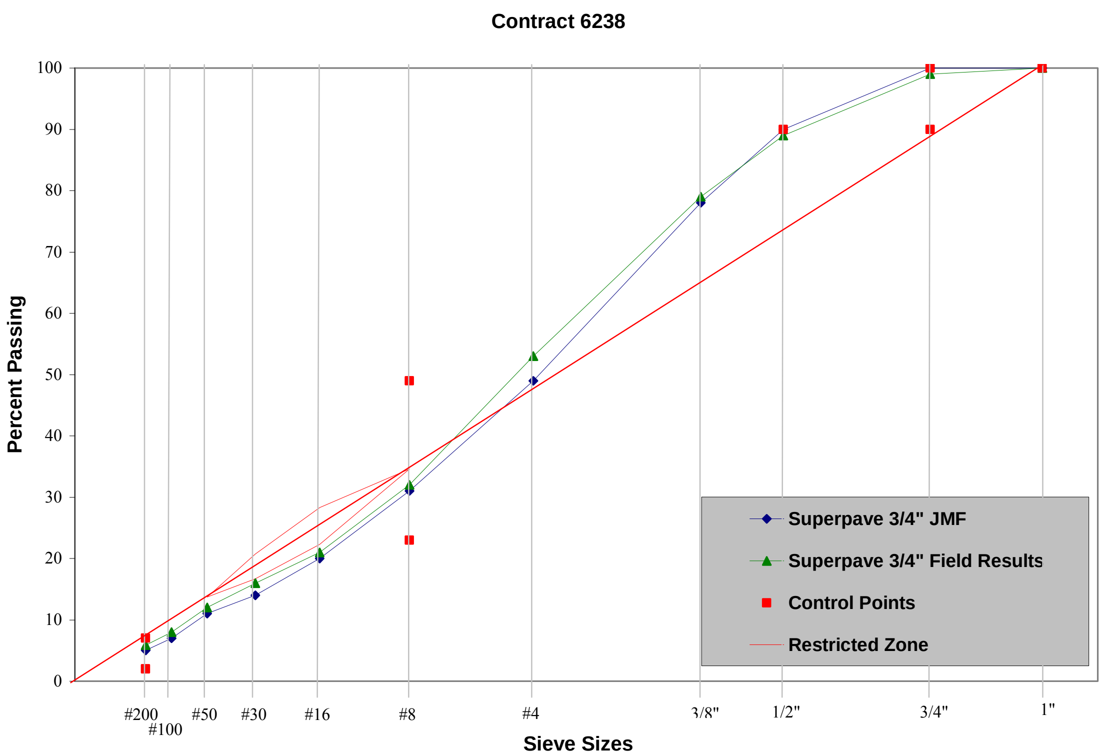
Control Points
Restricted Zone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Control Points
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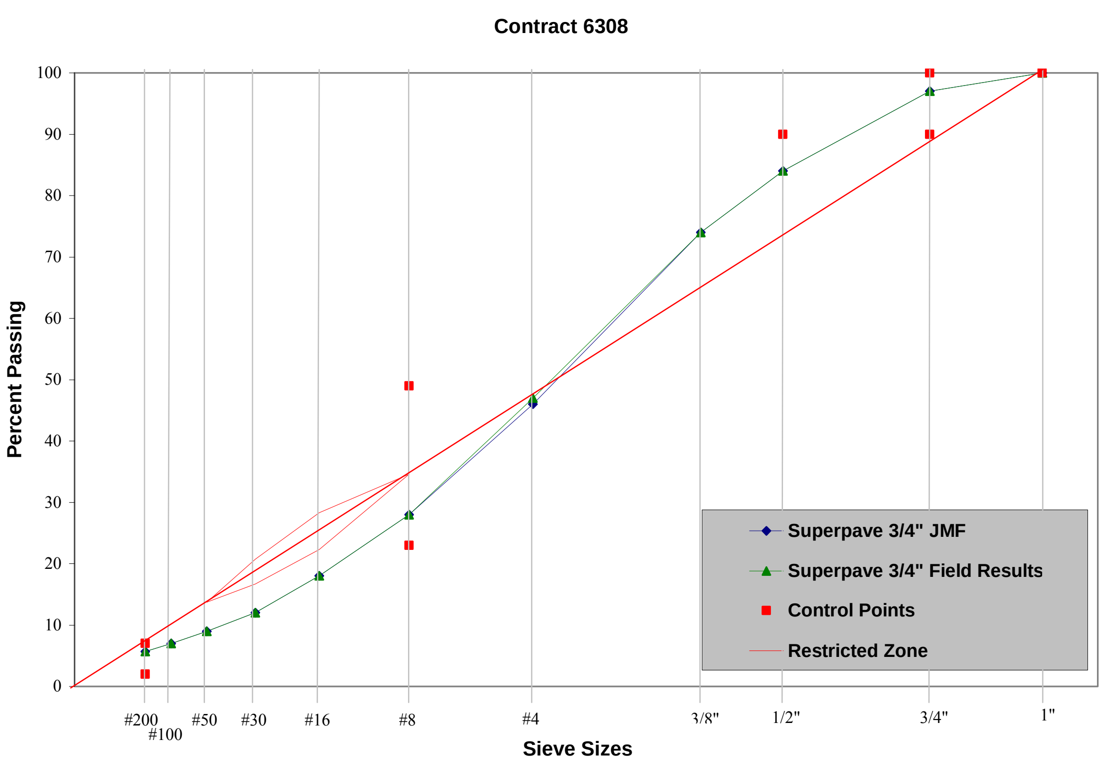
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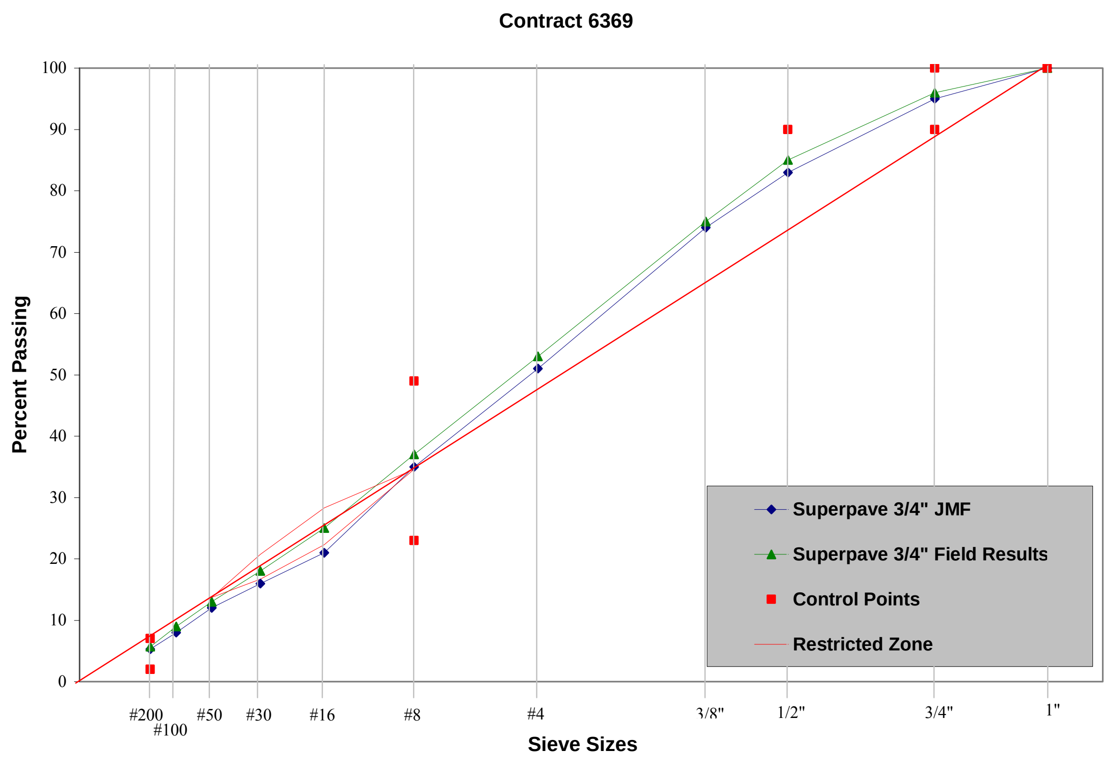
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Sieve Sizes
Superpave 3/4" JMF
Superpave 3/4" Field Results
Control Points
Restricted Zone

Appendix B

Section Photos

Figure 8. I-90, Vantage Bridge to Burke longitudinal cracking.

Figure 9. I-90, Vantage Bridge to Burke transverse and alligator cracking.

Figure 10. I-90, RR O'Xing to Adams Co. Line transverse crack and coarse surface texture.

Figure 11. RR O'Xing to Adams Co. Line flushing in wheel paths.

Figure 12. SR-17, Lind Coulee Br. to Vic. I-90 alligator cracking and coarse texture.

Figure 13. SR-17, Lind Coulee Br. to Vic. I-90 coarse texture.

Figure 14. SR-395, SR-17 to Connell & SR-260 to Adams Co. Line, longitudinal cracking.

Figure 15. SR-395, SR-17 to Connell & SR-260 to Adams Co. Line, alligator cracking.

Figure 16. SR-240, Stevens Dr. to SR-182, coarse texture.

Figure 17. SR-240, Stevens Dr. to SR-182, coarse texture.

Figure 18. I-82, W. Prosser I/C to Oregon State Line, longitudinal and alligator cracking.

Figure 19. I-82, W. Prosser I/C to Oregon State Line, surface texture.

Figure 20. SR-221, SR-14 to Prosser Hill, uniform texture.

Figure 21. SR-221, SR-14 to Prosser Hill, open texture near centerline.

Figure 22**.** I-82, Valley Mall Blvd. to Yakima R., transverse and alligator cracking.

Figure 23**.** I-82, Valley Mall Blvd. to Yakima R., fine texture in wheel paths and coarse texture outside of wheel paths.

Figure 24. Naches R. Br. to Valley Mall Blvd. transverse and alligator cracking.

Figure 25. Naches R. Br. to Valley Mall Blvd. coarse texture.

Figure 26. I-90, Mercer Slough to 128th Ave SE, open texture.

Figure 27. I-90, Mercer Slough to 128th Ave SE, transverse crack.

Figure 28. SR-167, 8th St to 15th St. SW Carryover 2000, raveling in wheel paths.