December 31, 2013

REPORT

CITY OF HAMILTON

Pavement and Materials Technology Review - Phase III

Submitted to:
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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) is pleased to present this draft report to the City of Hamilton (City) with the findings of Phase III a review of the pavement and materials technologies currently in use or intended to be used by the City. Golder previously carried out two reviews (Phase I and II) of the paving technologies and practices in use at the City and provided recommendations for updates to these technologies. In addition, as part of these reviews, Golder provided recommendations for implementation of new technologies and methods for addressing new issues in the paving practice. Following completion of the first two phases of the review, the City retained Golder to carry out a third phase of the review in accordance with our proposal dated March 11, 2013. As described in our proposal, the third phase of the Pavement and Materials Technology (PMTR) review was divided in to four main components listed below:

- Item 1 – Implementation of Phase I and Phase II of the PMTR and overview of the 2012 and 2013 pavement construction quality;
- Item 2 – Development of an updated pavement design matrix;
- Item 3 –Pavement Management System (PMS) review and recommendations for maintenance and rehabilitation alternatives as related to it; and
- Item 4 – Analysis of and recommendations for new paving technologies.

This draft report includes the findings of our review for each of the four items above and provides recommendations based on the findings.

2.0 ITEM 1 – IMPLEMENTATION OF PMTR PHASE 1 AND PHASE II AND OVERVIEW OF 2012 AND 2013 PAVEMENT CONSTRUCTION QUALITY

For Item 1 of this assignment, Golder reviewed the recommendations that were previously provided to the City during Phases I and II of this project to identify which of the recommendations had been implemented and whether they were producing favourable results. In order to identify the impact of the changes that have been implemented following the Phase I and II reviews, Golder obtained the Quality Control testing results from 2012 and 2013 to determine the quality and material being placed.

2.1 Phase 1 of PMTR

The first phase of the pavement and materials technology review was focused on construction quality including a review of the City's Quality Control (QC) and Quality Assurance (QA) practices. Also during Phase I, a field inspection was carried out for the performance of and construction practices used for longitudinal joints. Golder met with City staff during Phase I to take note of their primary concerns undertook a visual condition inspection of some of the typical pavement sections in the City.

The City of Hamilton has been recognized for years as the leader in innovations. It has a knowledgeable staff and good quality construction materials are available in the vicinity. Additionally, Experienced and good contractors are also available. However, during Phase I it was noted that the pavement were generally underperforming and that improvements in the QC/QA practices were required. The typical distresses that were observed included:

- Joint opening;
- Longitudinal, transverse and alligator cracking;
- Rutting;
- Potholes and patching;
- Poor utility cut restoration;
- Segregation; and
- Ravelling.

The QC/QA system was considered to inconsistent, contractor dependant, with poor quality enforcement and poor work preparation. QA testing was limited and inspection requirements were not fully understood as the QC/QA aspects in the specifications were not clear. This portion of the specifications was subsequently revised as part of Phase I of this project. Additionally, the required testing for QC/QA purposes was clarified, including the type of tests, the limits, the frequency of testing, reporting, acceptance and enforcement. Finally, training was provided to the City staff on the updates recommended in Phase I.

For the improvement of the longitudinal joints, recommendations were provided during Phase I and these included:

- Echelon paving;
- Application of infrared heaters on joints not paved in echelon;
- Use of warm mix asphalt technology; and
- Use of better construction practices for construction of regular cold joints.

2.2 Phase II of PMTR

Phase II of the pavement and materials technology review was focused on the following:

- Superpave asphalt mix technology;
- Asphalt pavement construction practices;
- Fine tuning QC/QC procedures;
- Upgrading specifications; and
- Warm mix asphalt technology.

During a meeting with City staff for Phase II of this project, the issues requiring most urgent attention were discussed. The issues identified by the City staff included the paving specifications, granular materials, aggregates, asphalt cement, asphalt mixes, pavement rehabilitation, pavement preservation and staff training.

The City decided to move to Superpave asphalt mixes in 2009. At first, about 50 percent of the projects in 2009 used Superpave mixes and then in 2010, 100 percent of the project incorporated Superpave mixes. The visual pavement condition inspection carried out during Phase II of the new Superpave mixes indicated typical problems identified by other authorities with such mixes, including, cracking occurring very early in the pavement life and aged, dry surface appearance with associated ravelling. This was addressed by providing recommendations for updates to the paving specifications including the minimum required asphalt cement content and these recommended updates were incorporated in the new specifications. Among other changes to the specifications, one of the major amendments recommended was the tightening of the HMA production limits, particularly for the asphalt cement contents. A detailed review of the QC/QA results, discussed in subsequent sections of this report, indicated that after the changes to the paving specifications were made, the quality of the produced HMA mixes was significantly improved.

The limestone aggregates that are available in the area were evaluated during Phase II, particularly in terms of their resistance to polishing.

Phase II also included further review of WMA technology for implementation in the City and Golder provided the City with technology recommendations and methods of addressing the mix design procedure.

Finally, during Phase II Golder provided the City staff with training on the following aspects:

- Superpave technology including WMA;
- Understanding the specifications including QC/QA procedures and requirements for construction inspection;
- Interpretation of the QC/QA testing results, including acceptance and enforcement; and
- Best construction practices including those specific to longitudinal joints.

2.3 Outcome of Phase I and Phase II Recommendations

2.3.1 Review of 2012 and 2013 Quality Assurance Testing Results

To evaluate whether the recommendations from Phase I and Phase II resulted in improved pavement performance, the QA testing results from paving in 2012 and 2013 were reviewed. These results clearly indicated that there has been a significant improvement in the quality of construction since the recommended changes were made and the staff training was provided. Figure 1 to 20 shows the distribution of testing results for the SP 12.5, 19.0 and 9.5 mm mixes. The results include the percent passing the 4.75 mm sieve and the 0.075 mm sieve, the asphalt cement content and the mean compaction. The distribution for the all the testing results from the QA testing carried out during 2010, 2011, 2012 and 2013 are included in Appendix A.

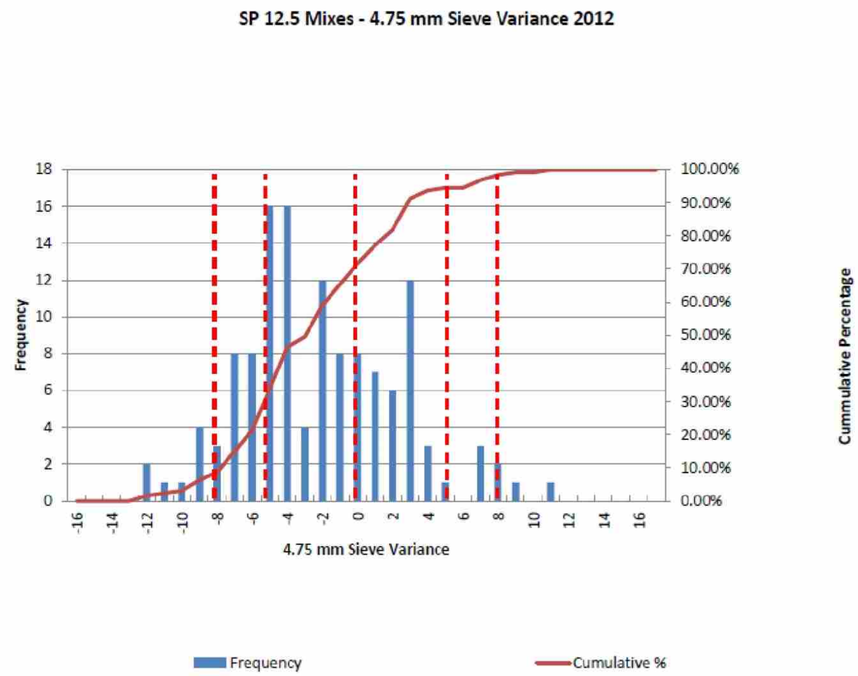

Figure 1: Variance for 2012 SP 12.5 Mixes for Percent Passing 4.75 mm Sieve

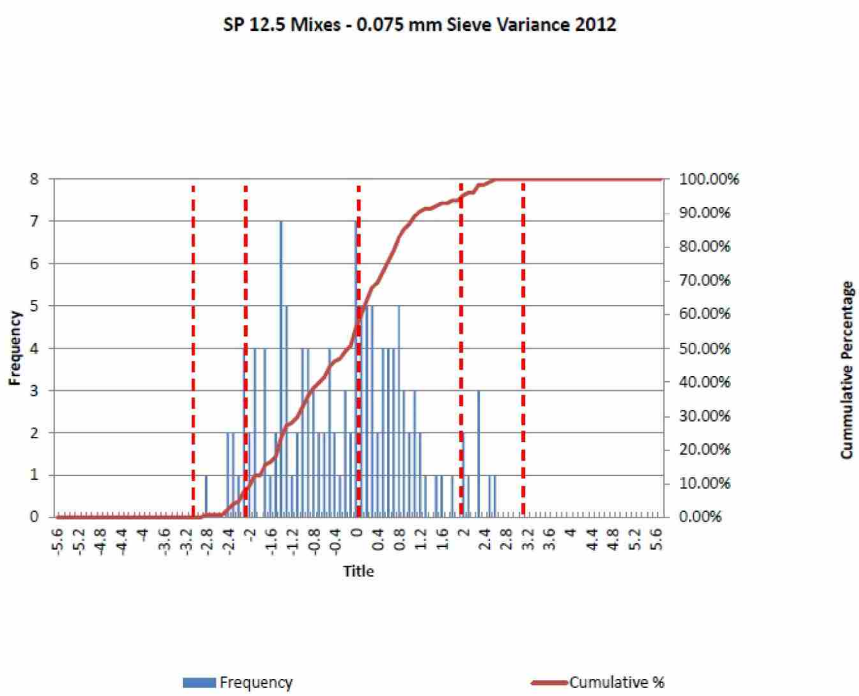

Figure 2: Variance for 2012 SP 12.5 Mixes for Percent Passing 0.075 mm Sieve

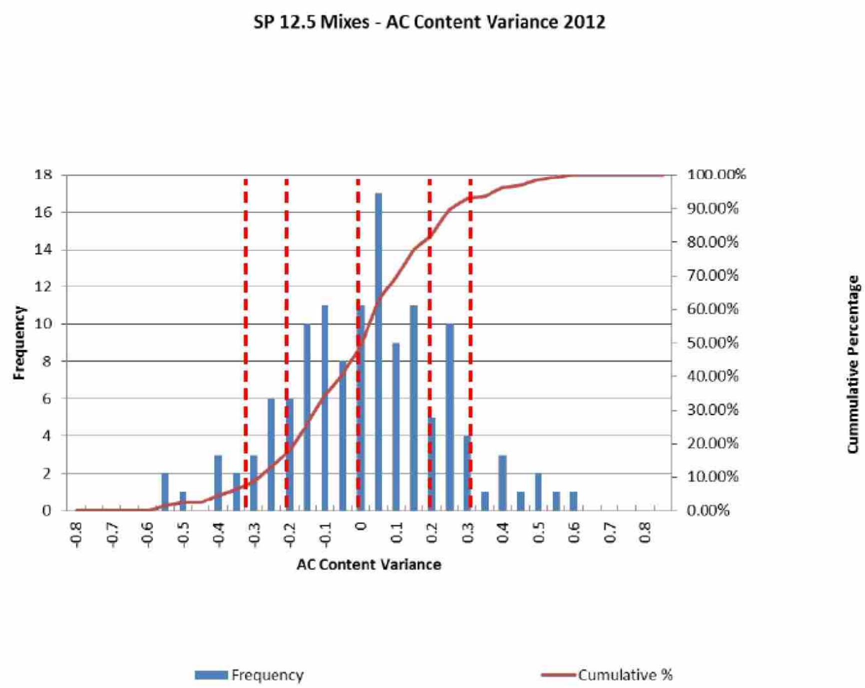

Figure 3: Variance for 2012 SP 12.5 Mixes for AC Content

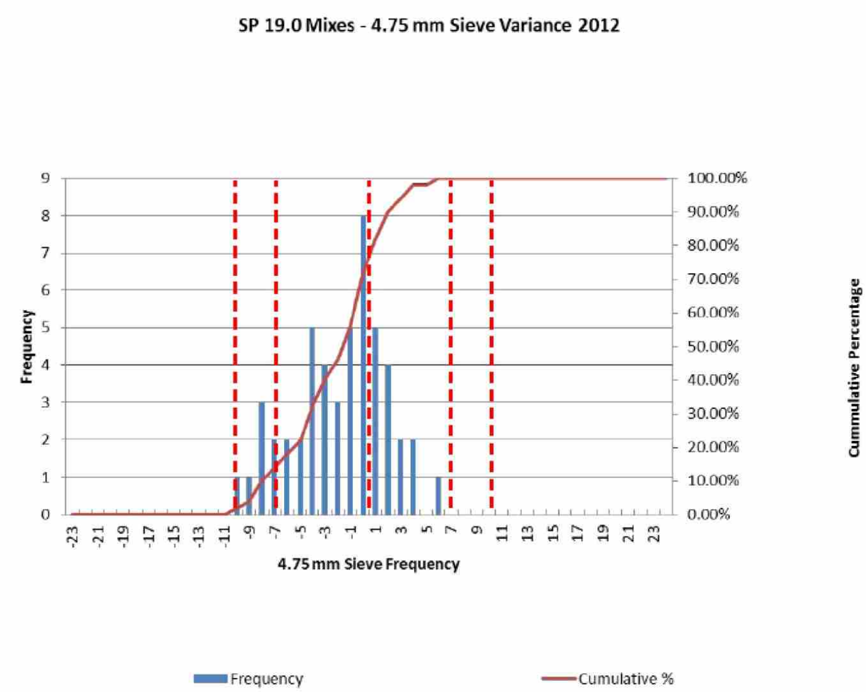

Figure 4: Variance for 2012 SP 19.0 Mixes for Percent Passing 4.75 mm Sieve

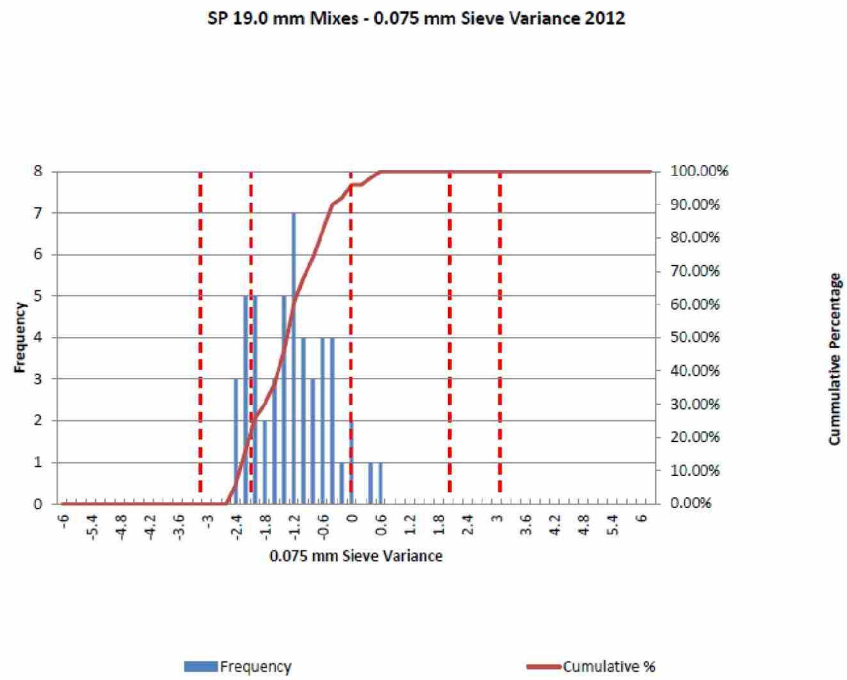

Figure 5: Variance for 2012 SP 19.0 Mixes for Percent Passing 0.075 mm Sieve

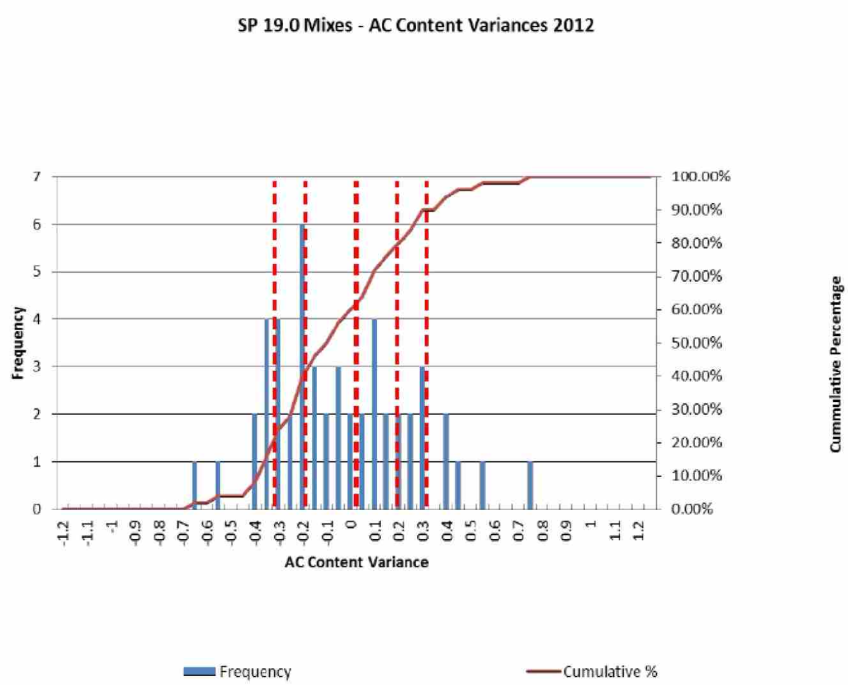

Figure 6: Variance for 2012 SP 19.0 Mixes for AC Content

SP 9.5 mm Mixes - 0.075 mm Sieve Variance 2012

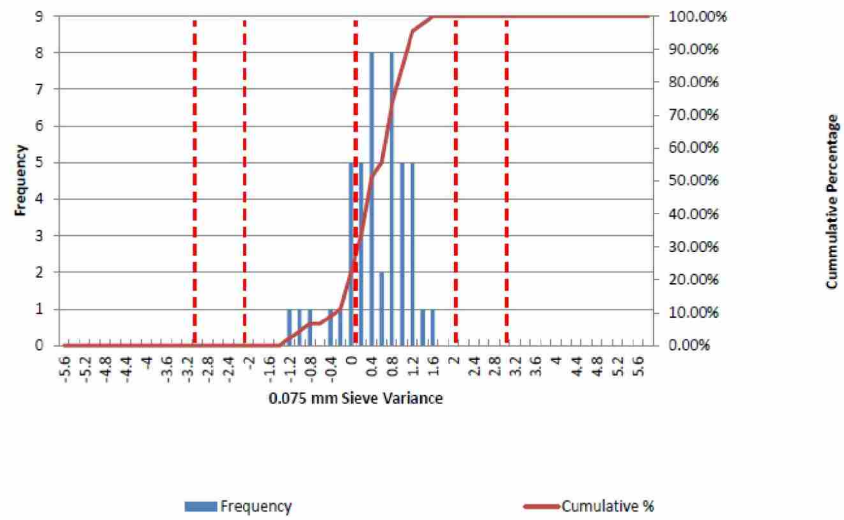

Figure 7: Variance for 2012 SP 9.5 Mixes for Percent Passing 0.075 mm Sieve

SP 9.5 Mixes - AC Content Variance 2012

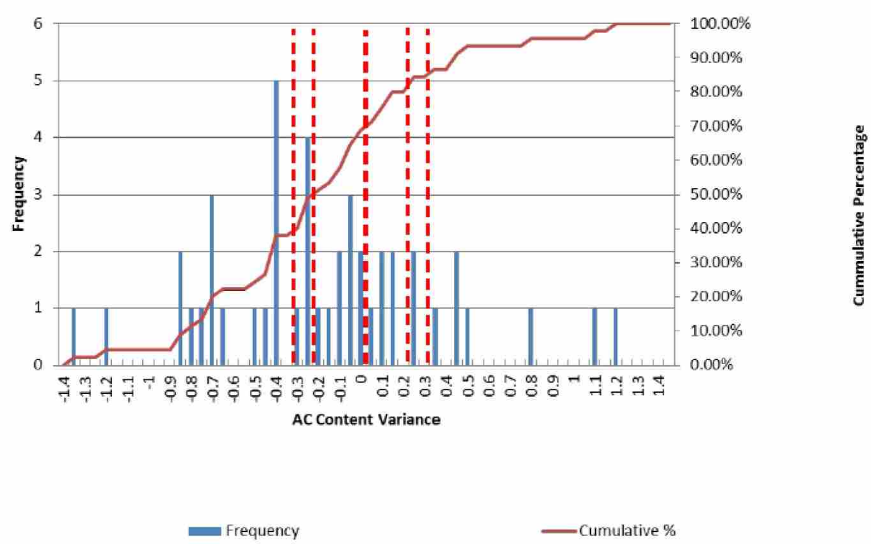

Figure 8: Variance for 2012 SP 9.5 Mixes for AC Content

GOL0007504

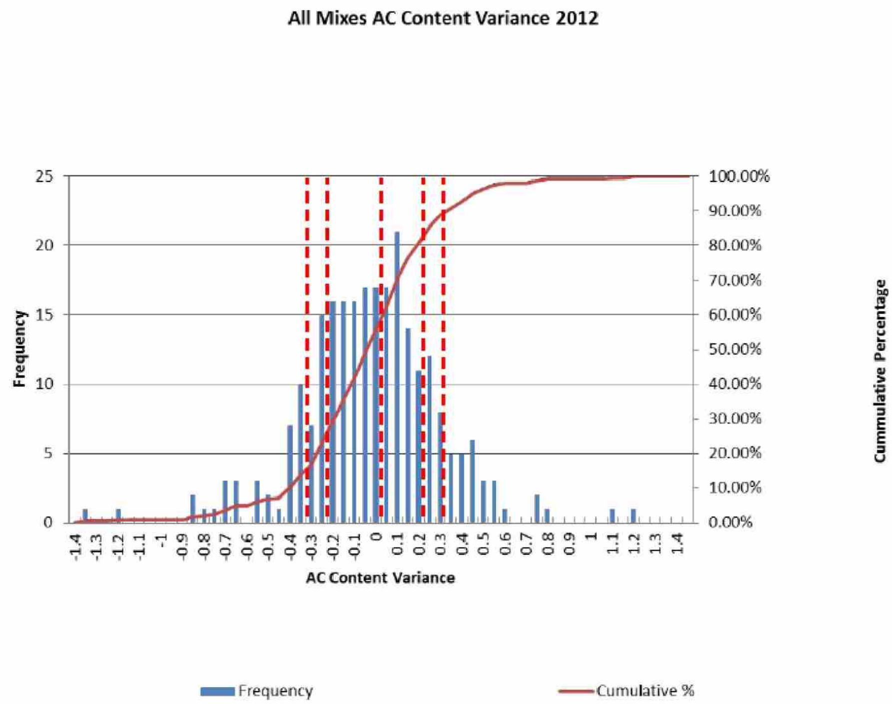

Figure 9: Variance for All 2012 Mixes for AC Content

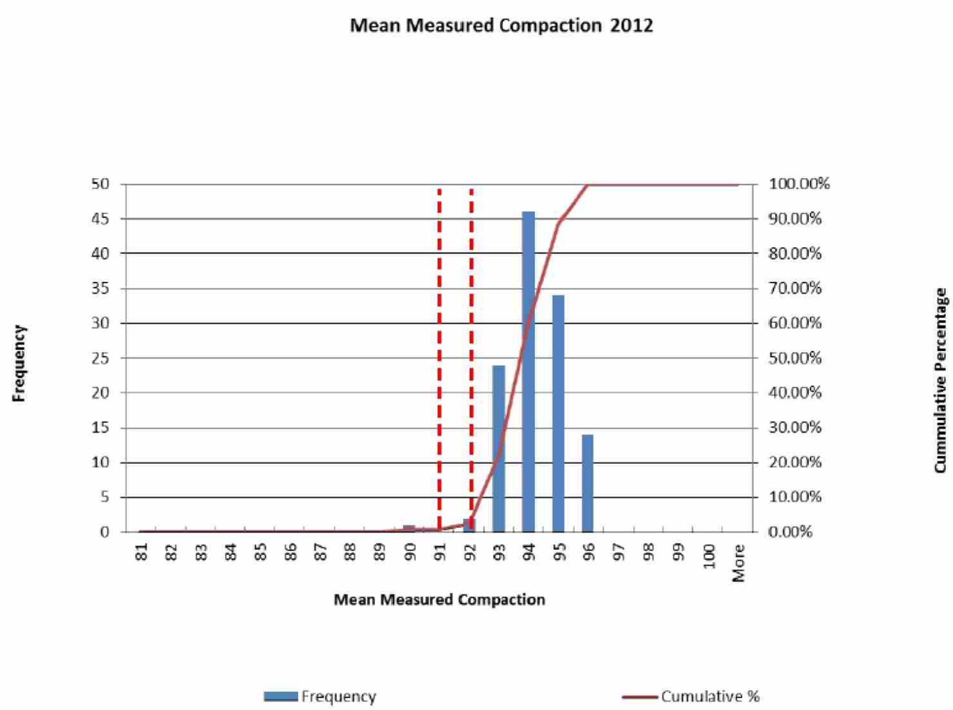

Figure 10: Mean Measured Compaction for All 2012 Mixes

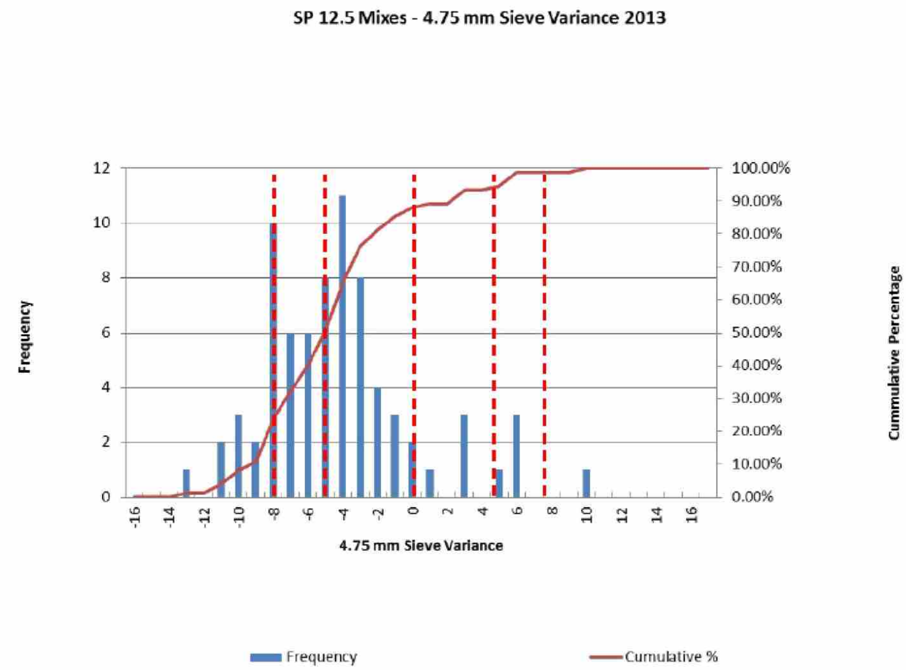

Figure 11: Variance for 2013 SP 12.5 Mixes for Percent Passing 4.75 mm Sieve

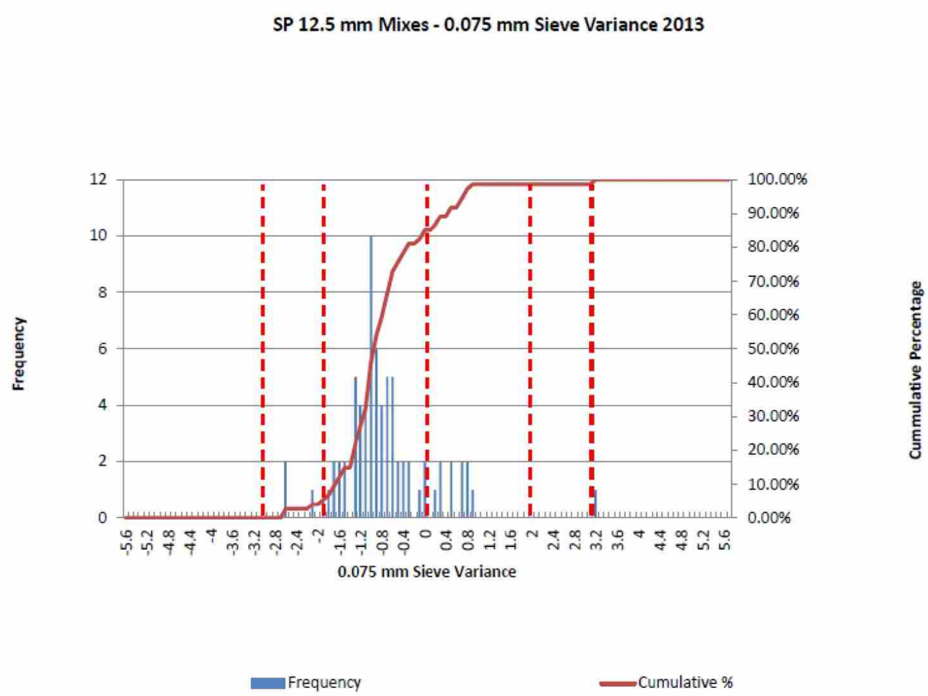

Figure 12: Variance for 2013 SP 12.5 Mixes for Percent Passing 0.075 mm Sieve

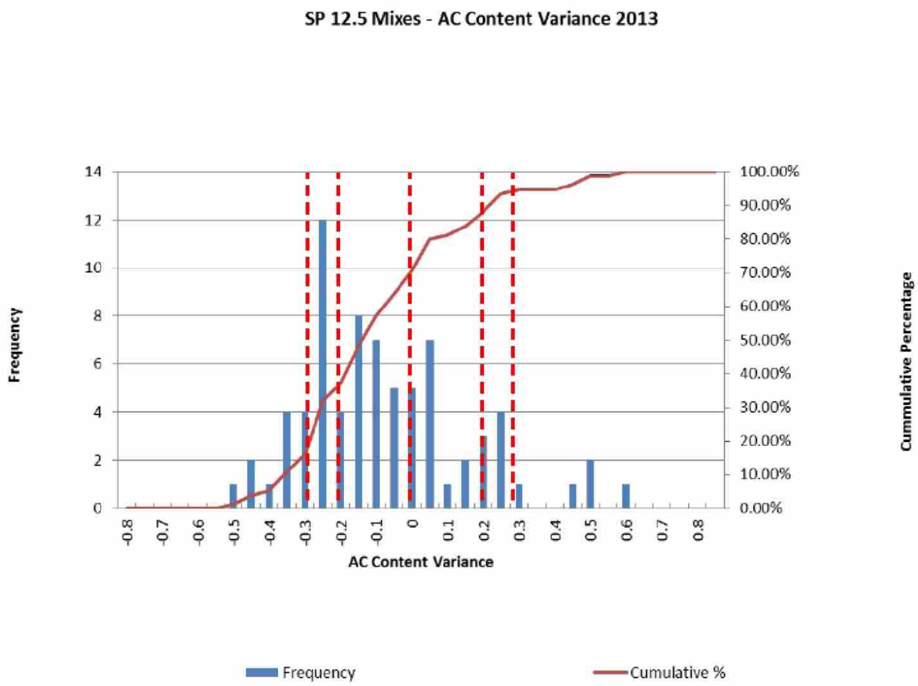

Figure 13: Variance for 2013 SP 12.5 Mixes for AC Content

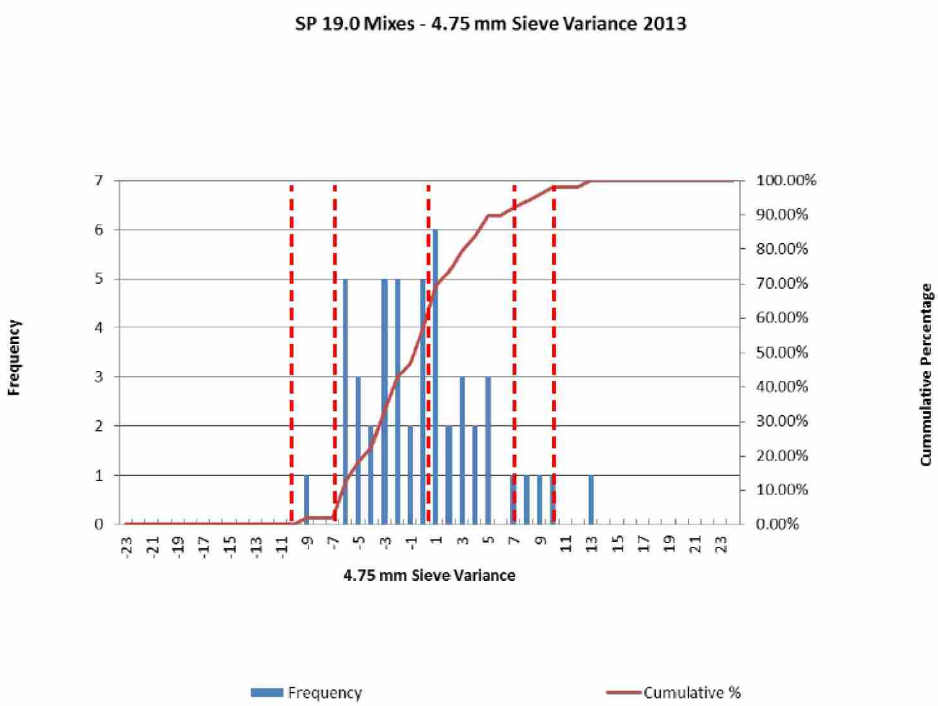

Figure 14: Variance for 2013 SP 19.0 Mixes for Percent Passing 4.75 mm Sieve

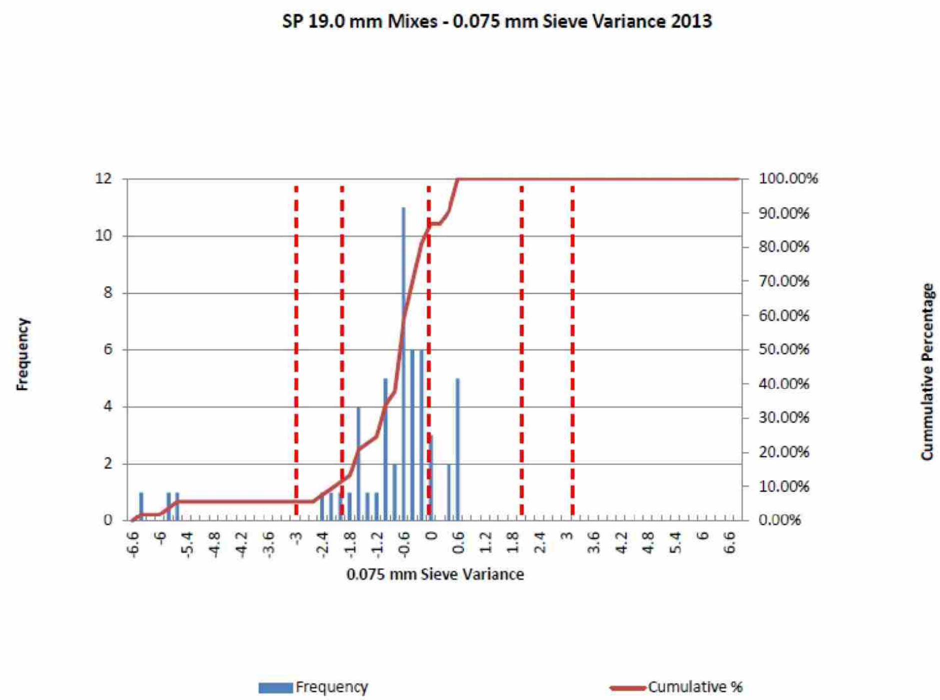

Figure 15: Variance for 2013 SP 19.0 Mixes for Percent Passing 0.075 mm Sieve

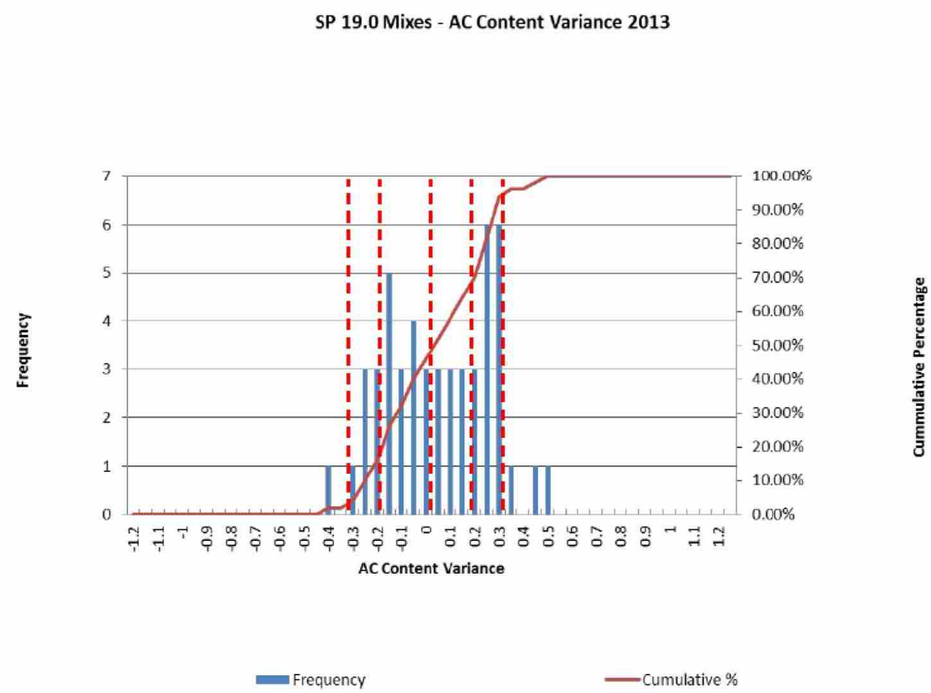

Figure 16: Variance for 2013 SP 19.0 Mixes for AC Content

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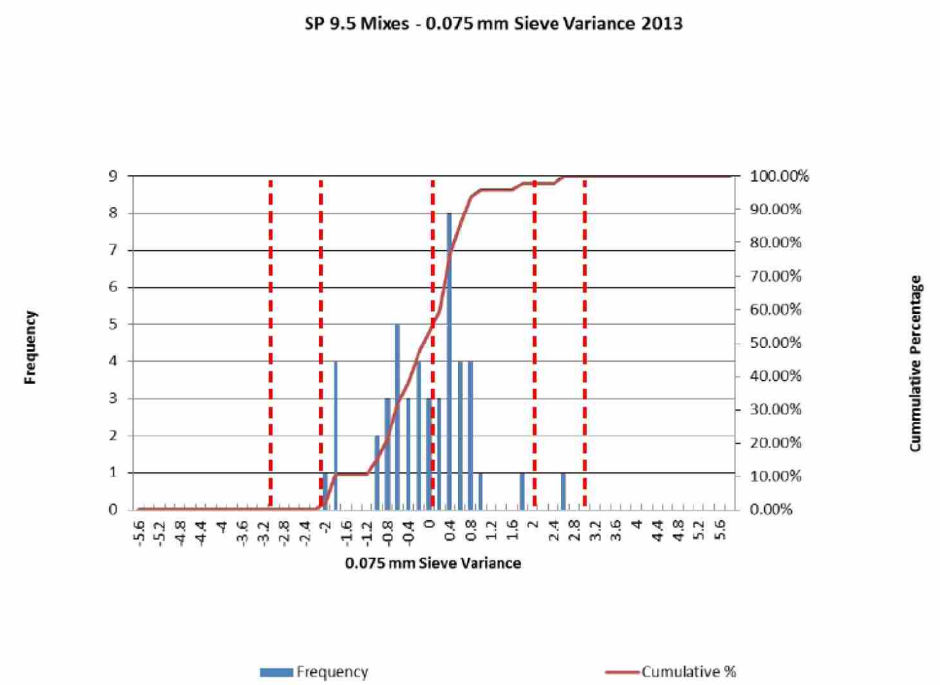

Figure 17: Variance for 2013 SP 9.5 Mixes for Percent Passing 0.075 mm Sieve

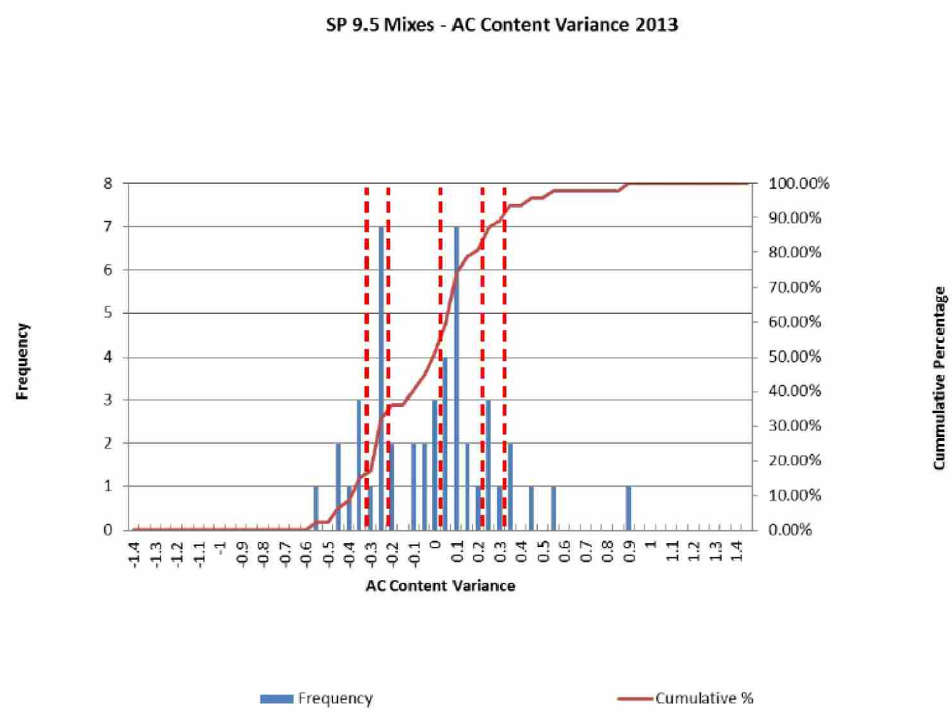

Figure 18: Variance for 2013 SP 9.5 Mixes for AC Content

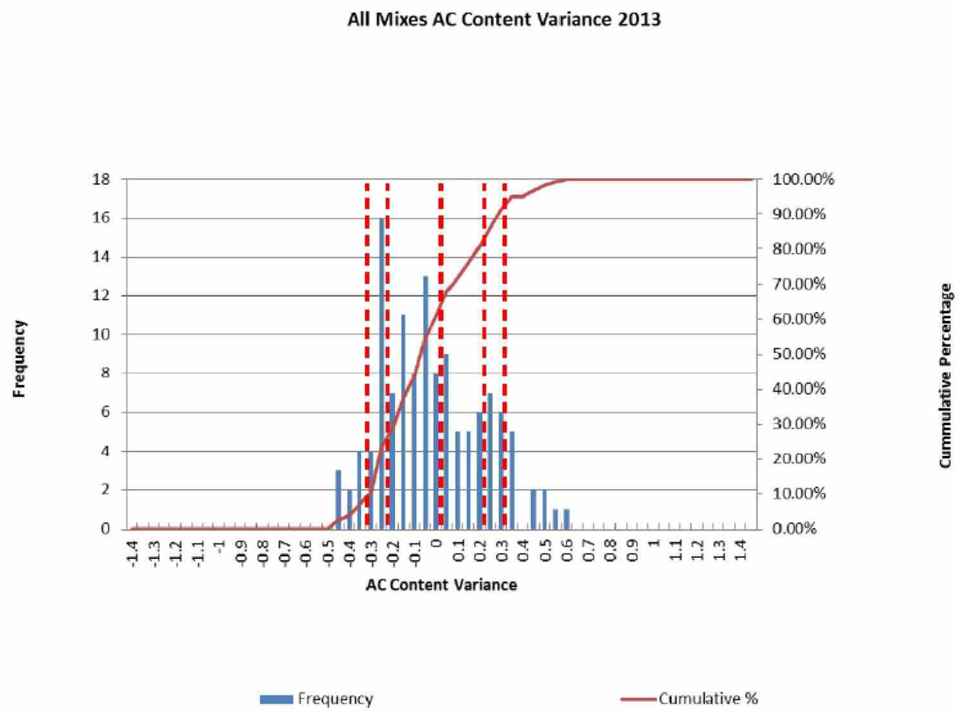

Figure 19: Variance for All 2013 Mixes for AC Content

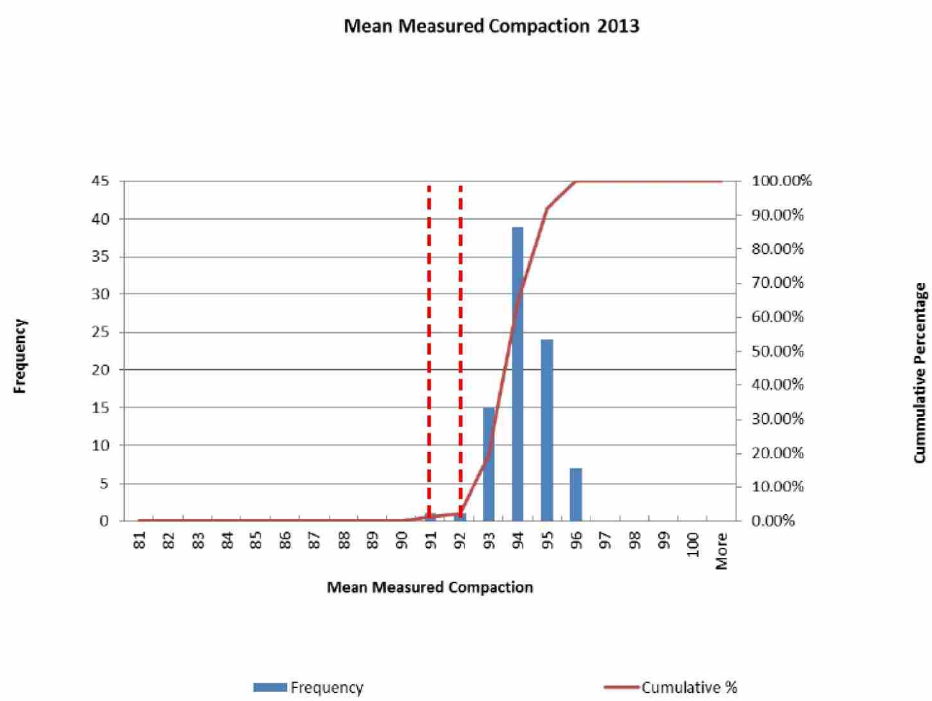

Figure 20: Mean Measured Compaction for All 2013 Mixes

It is very evident from the 2012 and 2013 results for the SP 12.5 and SP 19.0 mixes that there is significant improvement in quality as compared to the same mix types placed in 2010. The changes to the specifications are therefore considered to be effective ads resulted in improvement in the quality of material being produced in placed on the road in the City.

Since the distribution of results in 2012 and 2013 are very similar, it can be concluded that a plateau has been reached in term of quality improvement for the SP 12.5 and 19.0 mm mixes. However, for the SP 9.5 mm the testing results for 2012 and 2013 have seen no improvement since its first placement in 2011. Our review showed that some of the testing results from the SP 9.5 mixes are outside the borderline zone. This is likely due to somewhat limited experience that the contractors have with this particular type of mix. This mix is used only sporadically by other municipalities and by the MTO.

2.3.2 Pavement Visual Condition Inspection

In addition to reviewing the QA testing results, a senior pavement specialist from Golder carried a visual condition inspection of select pavement to evaluate their performance after the changes from Phase I and Phase II were implemented. The photographs taken during the December 9, 2013 condition inspection are included in Appendix B. The condition inspection clearly showed an improvement in the pavement surface quality when compared to the pavements constructed in 2009 and 2010. There was no early age ravelling visible in the 2012 of 2013 pavement. Also, the pavements constructed in 2012 and 2013 had generally good quality longitudinal joints.

Listed below are the roads that were inspected in 2013, the year the road was paved, and its overall condition in 2013 from the inspection.

- Barton Street – SP 12.5 mm mix paved in 2012 was in generally good condition;
- Huron Street – SP 9.5 mm mix paved in 2012 was in good condition;
- King Street – SP 12.5 mm mix paved in 2013 was in good condition and the longitudinal joints were of good quality;
- Mohawk Road – SP 12.5 mm mix with fibres paved in 2013 was in good condition;
- Mohawk Road – SP 12.5 mm mix without fibres paved in 2013 was in good condition;
- Lincoln M. Alexander Parkway – SP 12.5 mm mix paved in 2010 was observed to have some cracking that has reflected through the overlay.
- Red Hill Valley Parkway – Stone Mastic Asphalt (SMA) paved in 2007 was observed to have a number of top down cracks.
- Sherman Avenue – SMA with Manufactured Shingle Modifier (MSM) was observed to have extensive cracking and ravelling.

In addition to the above road sections, a number of pavement sections were inspected that were paved with surface course asphalt mixes containing steel slag. During the inspection all the sections containing steel slag were noted to have significant cracking of a very distinct pattern, as shown in the pictures in Appendix B.

3.0 ITEM 2 – DEVELOPMENT OF AN UPDATED PAVEMENT DESIGN MATRIX

For Item 2 of the review, the City provided Golder with the current design matrix that is used by City staff to specify the pavement structure that should be constructed on a particular roadway within the City's jurisdiction. Golder carried out a review of the existing design matrix and had a discussion with City staff to determine the primary issues encountered with the use of the existing matrix. The review findings were subsequently used to develop recommendations for updates to the current matrix.

3.1 Current Flexible and Rigid Pavement Design Matrix

The existing design matrix that is used by the City staff takes traffic volumes into consideration only based on the classification of a particular roadway. The City road network is divided into the following traffic categories:

- Local;
- Collector with no bus traffic;
- Collector with low bus traffic;
- Collector with high bus traffic;
- Commercial/Industrial and Minor Arterial; and
- Arterial.

It is understood that local roads are generally the residential road where the majority of the traffic is personal vehicles with the exception of a limited number of waste disposal truck; emergency vehicle and moving trucks. The City's collector road network would generally take significantly more heavy vehicle traffic than the local roads and is subdivided in to three categories based on the volume of trucks and busses. The highest traffic volumes, in particular heavy vehicle traffic, would have to be accommodated by the City's arterial roads, which are further subdivided based on bus and truck volume. The City also has a network of roads that are classified as commercial or industrial roads which generally has relatively lower volumes of traffic; however, a high percent of the traffic consists of heavy vehicles. Therefore, commercial/industrial roads are considered to be the same as minor arterial within the City's existing design matrix.

During the structural design analysis of a road, in addition to traffic volumes some of the other design inputs that are required are as listed below:

- Level of serviceability following the road construction;
- Level of serviceability indicating end of pavement service life;
- Reliability that a particular design will meet its service life;
- Deviation in the design inputs including traffic and material properties during construction; and
- Subgrade bearing capacity.

In general the first four design parameters listed above can be inferred based on the classification of a particular road. A road classification not only implies the traffic levels but also indicates the level of service that the travelling public would expect and the importance of the road within the overall network. For example, a local road may be constructed to lower levels of reliability and lower initial levels of serviceability than a collector. Similarly, a local road as compared to a collector may have lower values of terminal serviceability that indicate end of the service life, due to the slower traffic speeds. Since the existing matrix provides different designs for the various road classifications in the City, it can be assumed that the first four parameters listed above have been somewhat reflected.

For the subgrade bearing capacity, the classification of roadway is of no particular significance and the selected value is based primarily of the soil type on which the pavement will be constructed and the in place ground water and drainage conditions. Our review of the existing design matrix showed that no particular consideration is given to the different soil conditions that may be encountered in the City, when the structural design of a pavement is being selected. In discussion with the City staff it was noted, that the lack of subgrade soil conditions in the design analysis, was one of the primary shortcomings of the existing design matrix.

The existing design matrix includes structural design recommendations only for pavement reconstruction, generally with two options; flexible pavement and composite pavement (asphalt on concrete). In the case of arterial roads, an additional reconstruction option with a rigid pavement cross-section is also included in the matrix. In addition to the reconstruction pavement structural designs, the matrix also includes the type of asphalt mix that should be used for a resurfacing; however, it does not include the recommended thickness. Based on discussion with the City staff, Golder understands that the thickness for a resurfacing is generally based on the information included in the City's Pavement Management System (PMS) and does not take into consideration the structural capacity of the existing pavement layers.

Also noteworthy is that the existing design matrix only includes reconstruction designs as a major rehabilitation of an existing roadway and resurfacing as a minor rehabilitation. The matrix does not give any consideration to other rehabilitation strategies, such as those listed in section 4.2.2 of this report. It is important to note that in order to develop suitable structural designs for rehabilitation of existing pavements when portion of the existing pavement is going to be salvaged, it is critical to know the quality and thickness of the existing pavement layers.

In addition to the six main road classifications listed above, the current design matrix also includes a separate road classifications for the Lincoln M. Alexander Parkway (Linc) and the Red Hill Valley Parkway (RHVP). For both these facilities only a resurfacing option was provided and no major rehabilitation alternative was included in the design matrix.

3.2 Recommendations for Design Matrix Updates

In order to update the existing design matrix, Golder carried out a structural design analysis using the American Association of State Highway and Transportation Officials (AASHTO) 1993 "Guide for Design of Pavement Structures". During discussion with the City staff it was brought to Golder's attention that the primary subgrade soil types in the Hamilton area are as listed below:

- Clays;
- Sandy Silts to Clayey Silts; and
- Shallow bedrock.

Using the above information, a structural design analysis was carried out by Golder for the three different soil types and for the six different road classifications. When carrying out the design analysis it was assumed that the structural design in the current matrix were developed for the clay subgrade soils. This information was subsequently used to determine the approximate Equivalent Single Axle Loads (ESALs) that a pavement in each particular road classification may have to accommodate. It is noteworthy, that for the structural design analysis the three different collector categories were combined into one and the two arterial categories were also combined to a single classification. Therefore, for the thickness design analysis, the six road classifications were reduced to three classifications. However, for the classifications that were combined for the thickness analysis, the differences in the anticipated traffic loading were accounted for in the quality of the materials being utilized to construct the pavement layers, in particular the asphalt layers.

Table 1 below shows the parameter values that were used for the design analysis for the local roads. Similarly, Table 2 and 3 show the parameter values that were used for the structural design analysis for the collector and arterial roads, respectively.

Table 1: Parameter Values for Structural Design Analysis for Local Roads

Parameter		Value
Maximum Cumulative Equivalent Single Axle Loads		2,000,000
Initial Serviceability		4.5
Terminal Serviceability		2.3
Reliability		85%
Standard Deviation		0.45
Subgrade Resilient Modulus for Clay Soils		30 MPa
Subgrade Resilient Modulus for Clayey Silt Soils		25 MPa
Subgrade Resilient Modulus for Shattered Bedrock		70 MPa
Structural Coefficient	Hot Mix Asphalt	0.44
	Granular A	0.14
	Granular B, Type II	0.12
Drainage Coefficient	All Layers	1.0

Table 2: Parameter Values for Structural Design Analysis for Collectors

Parameter		Value
Maximum Cumulative Equivalent Single Axle Loads		4,500,000
Initial Serviceability		4.5
Terminal Serviceability		2.5
Reliability		90%
Standard Deviation		0.45
Subgrade Resilient Modulus for Clay Soils		30 MPa
Subgrade Resilient Modulus for Clayey Silt Soils		25 MPa
Subgrade Resilient Modulus for Shattered Bedrock		70 MPa
Structural Coefficient	Hot Mix Asphalt	0.44

Parameter		Value
	Granular A	0.14
	Granular B, Type II	0.12
Drainage Coefficient	All Layers	1.0

Table 3: Parameter Values for Structural Design Analysis for Arterials and Commercial/Industrial Roads

Parameter		Value
Maximum Cumulative Equivalent Single Axle Loads		11,000,000
Initial Serviceability		4.5
Terminal Serviceability		2.5
Reliability		90%
Standard Deviation		0.45
Subgrade Resilient Modulus for Clay Soils		30 MPa
Subgrade Resilient Modulus for Clayey Silt Soils		25 MPa
Subgrade Resilient Modulus for Shattered Bedrock		70 MPa
Structural Coefficient	Hot Mix Asphalt	0.44
	Granular A	0.14
	Granular B, Type II	0.12
Drainage Coefficient	All Layers	1.0

Using the parameter values in Tables 1, 2 and 3, the designs in the existing matrix were used to determine the ESALs that could be accommodated for a clay subgrade. Subsequently, these ESAL values were then used to generate equivalent designs for the clayey silt and shallow bedrock subgrade conditions. The updated design matrix is included Appendix C and the detailed design sheets are included in Appendix D.

Listed below are the primary updates that are recommended to be made to the current design matrix used by the City and these updates are generally reflected in the updated matrix in Appendix C.

- For each of the road classifications, different structural design should be provided for the different possible subgrade soil conditions that may be encountered.
- For the composite and rigid pavement structures, the placement of a minimum of 150 mm of Granular A beneath the concrete should be considered for all road classifications. Placement of granular layer less than 150 mm will likely result in a non uniform base support for the concrete placement which in turn would result in diminished performance life. For softer subgrade soils and subgrades with more non uniformity and undulations, granular layer thicknesses should be increased from the minimum 150 mm to 200 mm or more.
- The SP 9.5 mm mixes should be limited only to local roads and the SP 12.5 mm mixes should be used as the surface course mix for all other road classifications (collectors and arterials). The SP 12.5 mm mixes generally have increased frictional and durability properties as compared to the SP 9.5 mm mix. Also, as indicated in the previous sections of this report, the results of the SP 9.5 mix testing are much poorer than

those for the SP 12.5 mm mix. This is likely Due to limited experience of the local contractors with this mix type as it is used only sporadically by other municipalities and the MTO.

- For the arterial roads where a rigid pavement structure is intended to be placed, the Granular A thickness should be a minimum 250 mm. The thickness of the Granular A layer should be increased in the case of soft subgrade or subgrades that are variable in quality and surface profile.

It addition to the above, we would recommend that facilities such as the Linc and the RHVP should not be included in the standard design matrix for determination of suitable rehabilitation strategies. Due to the traffic volumes of these facilities and their importance within the network, the rehabilitation strategy should be determined on an individual basis following a proper field investigation and detailed design. This process will ensure that the most economically feasible rehabilitation strategy is selected for these facilities which will increase the reliability of adequate performance of the rehabilitated pavement.

4.0 ITEMS 3 – ANALYSIS OF NEW PAVING TECHNOLOGIES

This section of the report presents the findings of the analysis carried out for select issues and emerging technologies in the paving industry. Also presented are our recommendations for the implementation of the new technologies and how to address the issues. Listed below are the items that are discussed and addressed in the following sections.

- Use of asphalt mixes that incorporate a high proportion of Reclaimed Asphalt Pavement (RAP);
- Use of Asphalt Cement (AC) incorporating Engine Oil Residue (EOR); and
- Preventative treatments of in-place asphalt surfaced pavements.

4.1 Asphalt Mixes with High Proportion of RAP

The current specifications at the City of Hamilton allow for the incorporation of RAP in both the surface and binder courses with the following restrictions being applied:

- Maximum 15 percent by mass in Superpave surface course asphalt mixes;
- Maximum 30 percent by mass in Superpave binder course asphalt mixes;
- If RAP proportion in Superpave binder course asphalt mixes are proposed to be between 30 and 50 percent, then written approval must be obtained by the Contract Administrator for the mix design and AC grade to be used in the mix;
- RAP proportions exceeding 50 percent are not allowed for any asphalt mixes; and
- RAP is not permitted in certain surface course mixes that are designated by facilities with high traffic loading.

There are a number of benefits to the incorporation of RAP into new asphalt mixes with the primary benefits being the reduced consumption of a virgin material, reduction in the proportion of milled materials being sent landfills and potential for significant cost savings. However, in spite of the significant benefits, there are still concerns with the higher use of this material which in turn requires regulation. The primary concern with the use of RAP in new asphalt mixes is the impact on the performance of the asphalt once it is placed in the field and is subjected to traffic loading and environmental conditions.

4.1.1 Background

In Ontario, the usage of RAP in new asphalt mixes was started by the Ministry of Transportation of Ontario (MTO) in 1978 [Lynch, Evers, 1981] and by 1991 and the usage RAP had become common practice in much of Canada. In the United States (US), RAP is allowed to be used in the binder course mixes by all of the individual states; however, a small proportion of the states do not allow the use of RAP in the surface course asphalt mixes. On the contrary, in a survey conducted by the North Carolina Department of Transportation, on behalf of the American Association of State Highway and Transportation Officials, in 2009 it was identified that the actual RAP usage by Contractor typically was only about 50 percent of the allowable limit in the specifications [Copeland, 2011]. It is worth noting, that the survey respondents were primarily in the United States with one Canadian provincial transportation agency also responding to the survey (Ontario) [Copeland, 2011].

To date a large amount of laboratory and field testing has been carried out to determine the impact of the use of RAP into new HMA. The results of this testing generally indicates that the addition of RAP into HMA mixes results in an increase in the stiffness (modulus) of the mix. This increased stiffness of HMA mixes in turn may result in concerns with the cracking endurance and ravelling potential of mixes containing large amounts of RAP. There has been a significant amount of research undertaken to determine whether asphalt mixes containing RAP have diminished cracking resistance as compared to virgin mixes. However, the majority of this research has produced contradictory findings. It is critical to note that even for virgin asphalt mixes, the primary criteria that governs the cracking performance of a mix is carrying out a proper mix design as per the specified procedures.

4.1.2 Concerns with Use of RAP

In spite of all the benefits of the introduction of RAP in to new asphalt mixes, both the owners as well as the contractors have concerns with using higher RAP percentages into the mixes. In a report prepared for the Federal Highway Administration (FHWA) in 2011, Audrey Copeland (Copeland) identified that primary concerns for the owners are as follows [Copeland, 2011]:

- Quality and field performance of the high RAP mixes;
- Quality and consistency of the RAP itself;
- Extent of blending between virgin and RAP binder and the grade to be utilized for the virgin binder;
- Mix design procedures and volumetric requirements for high RAP mixes; and
- Use of polymer modified AC in mixes containing high RAP content.

Copeland also identified the main concerns that Contractors had incorporation of higher RAP contents in to their asphalt mixes and they were as follows:

- Higher RAP contents are not allowed by the specifications;
- Control of RAP stockpiles;
- Dust and moisture content of RAP; and

- Increased quality control requirements for mixes containing higher RAP contents.

From the list of concerns above it can be concluded that the primary issues with the use of higher RAP contents in HMA are as listed below, regardless of whether you are a contractor or an owner.

- Quality of the RAP may be variable both in terms of gradation, quality of the aggregates and quantity and quality of asphalt cement. This in turn makes it difficult to design a mix containing RAP and furthermore once the mix is designed it may be difficult to produce a consist product.
- RAP as a material tends to behave differently in a mix as compared to virgin aggregates or the virgin AC. The RAP consists both of aggregate and aged asphalt binder and therefore a significant amount of debate occurs regarding the extent of blending between the new and aged AC. The extent of blending would impact the mix design procedure both in terms of the amount of AC that is required in the mix as well the actual grade of the blended AC in the mix.
- The aged binder in the RAP combines with the new AC in the mix which results in the new mix typically having an increased stiffness as compared to an equivalent mix being produced without any RAP. This increased stiffness implies that mixes that contain RAP will react differently to an applied load than mixes without RAP which impacts the performance in the field.

In order to asphalt mixes containing RAP to be considered suitable for use, they have to provide equivalent or better performance than the alternative virgin mixes. Additionally, the cost of these mixes has to be lower or at least comparable to mixes with RAP. Through numerous field placement and long term performance records, it has generally been concluded that asphalt mixes containing lower proportions of RAP provide adequate field performance and are generally more economical than their virgin mix counterparts. As mentioned previously, the proportion that is considered to be generally acceptable is dependant on the depth of the mix in the pavement structure and, to some extent, on the experiences of the agency in question. In Ontario, the use of binder course mixes with up to 30 percent and surface course mixes with 15 percent RAP is considered to be acceptable. In the case of both surface and binder course mixes in Ontario that contain RAP, generally the mix design procedure currently in use does not evaluate the properties of the blended binder and whether it meets the requirements of the specification.

4.1.3 Best Practices for Increasing the Proportion of RAP in HMA

In the recent research carried out by the National Center for Asphalt Technology at Auburn University, it was concluded that although asphalt mixes containing higher RAP proportions (up to 45 percent) did indeed result in mixes that were stiffer than the equivalent virgin mixes, the actual infield performance in terms of cracking due to load application was not dramatically impacted. However, it is critical to recognize that in order for the mixes to provide adequate performance proper mix design need to be carried out, including selection of the appropriate asphalt cement grade and amount that should be utilized in the mix. The following sections provide some of the practices that are critical to ensure that a good quality and consistent asphalt mix can be produced while incorporating higher proportion of RAP.

4.1.3.1 RAP Processing and Stockpiling

The processing and stockpiling practices for RAP are critical in ensuring a good quality and consistent product that can be successfully incorporated into a new asphalt mix. There are a number of different sources from which a contractor obtains their RAP and each source results may be a different product, in terms of gradation and properties. When looking at incorporating higher percentages of this material into new mixes it is important to know when material from different sources can be combined. RAP material obtained from surface course layer of a particular roadway, versus the binder course layer, contains aggregates that are typically of superior quality and have improved resistance to polishing. Therefore, this RAP would be more suitable to be added to a new surface course asphalt mix. Similarly, RAP obtained from an existing high traffic volume roadway would typically have aggregates with better physical properties that those obtained from local roads. In addition to separating RAP from different layer sources, another method of ensuring that the aggregates in the RAP used in a particular mix are of good quality is to specify the exact source of the RAP, e.g. from a particular highway and particular lift. This type of RAP is called “certified RAP”.

In addition to the aggregate quality, different RAP sources may also result in different gradation. When RAP is milled from an existing roadway it typically results in final product with a relatively uniform distribution of particle sizes. Conversely, other sources of the material may results in large chunks or slabs of asphalt that need to be further processed through crushing. Due to the milling and crushing operations that are used to produce RAP, the material may tend to have a relatively large proportion of fine material and in particular a larger percentage of dust. Therefore, it may relatively difficult to incorporate large percentages of RAP in new HMA as the combined aggregates may not meet the gradation requirements. In order to incorporate larger proportion of this material in to a mix it is critical that the RAP be separated in to different fractions. Similar to virgin aggregate stockpiles, at the minimum the RAP should be divided in to a coarse (retained on the 4.75 mm sieve) and fine fraction (passing the 4.75 mm sieve). Depending one the equipment available to the contractor and their experience and RAP application, the RAP may be further fractioned into other sizes. It is also important to note that fractioning the RAP significantly simplifies the mix design process and reduces time and effort required.

Due to the AC coating the aggregate particles in RAP, the material has a tendency to have a higher moisture content in the stockpiles and it is critical that during production of HMA all the moisture is removed. Also, due to the fact that RAP generally tends to be a construction and demolition reclaimed material, there is an increased potential for the presence of debris and deleterious material in the stockpiles. As the proportion of RAP in new HMA is increased, it becomes exceedingly important that the stockpiles of this material are kept as clean as possible.

4.1.3.2 Mix Design for High RAP Asphalt Mixes

The mix design procedures for asphalt mixes containing RAP is generally similar to that for virgin mixes. During the mix design procedure the RAP is considered to be one of the aggregate sources which has to meet the physical property requirements of the other aggregates. Additionally, the bulk specific gravity for the RAP needs to be determined and the gradation. It is important to note that the bulk specific gravity of the RAP is a particularly difficult property to determine in the laboratory due to the fact that the AC on the RAP aggregates needs to be removed. The RAP is then combined with the other aggregate sources in order to meet the gradation and voids in mineral aggregate requirements for the mix. Additionally, during the mix design procedure when RAP is being added to a mix, the amount of virgin AC that is added to a mix takes into consideration the contribution of the AC from the RAP.

In addition to the above, when incorporating high percentages of RAP in to a new asphalt mix, consideration needs to be given to the grade of the virgin AC that will be used in the mix. The grade of the virgin AC that is used in the mix needs to be such that once the virgin and aged AC (from the RAP) combines, the required grade for that mix is achieved. At lower proportions of RAP, the amount of aged AC being contributed from the RAP is

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not sufficient enough to have a significant impact on the grade of the combined AC in the mix. However, as the percentage of RAP increases in the mix, there is an increased stiffening effect from the aged binder to the combined binder in the mix. Therefore, in general the grade of the virgin binder would have to be softer (lower high temperature grade) for a mix incorporating large quantities of RAP than what is required for the same mix containing only virgin material. If the Superpave mix design system is used for asphalt mixes, then AASHTO M323 includes guidelines for the selection of virgin binder grade to be used depending on the amount of RAP that is included in the mix as shown in Table 1.

Table 4: Guidelines for Virgin Binder Grade Selection from AASHTO M323 [AASHTO, 2010]

Proportion of RAP	Recommended Virgin Asphalt Binder Grade
Less than 15%	No change in binder grade selection
Between 15% and 25%	Select virgin binder one grade softer than normal (e.g. select PG 52-34 if PG 58-28 would normally be used)
Greater than 25%	Develop a blending chart and follow recommendations of blending chart

Ranges that have are shown in Table 1 for the amount of RAP in a mix that would trigger changing the grade of the virgin binder, are just guidelines and have been altered by a number of agencies in the United States based on local experience. In Ontario, the current practice is to generally not change the virgin AC grade for binder course mixes containing up to 30 percent RAP. For the surface course mixes, only up to 15 percent RAP is allowed to be included and no adjustment is required to the grade of the virgin AC to be used in the mix.

The development of a blending chart is a process that requires a significant amount of effort and equipment. This in turn can significantly reduce the financial savings that may have been achieved by using larger proportion of RAP as compared to virgin material. In general, the development of a blending chart involves characterization of the critical temperatures for the aged asphalt cement extracted from the RAP and then using an equation to determine the maximum allowable proportion of RAP that can be used in a mix (if the virgin AC grade is known) or determine the required grade for the virgin AC (if the amount of RAP to be added to the mix is known). It is also important to note that the method of blending charts to determine the grade for the virgin AC or to determine the maximum RAP percentage allowed, assumes that there is a 100 percent mixing between the aged binder in the RAP and the virgin AC. Although previous research has concluded that there is likely a significant amount of mixing that does occur, this research is generally inconclusive regarding whether there is complete mixing.

Recent research carried out by the National Center of Asphalt Technology at Auburn University carried out characterization of a blend of virgin AC of known grades and aged AC from two RAP sources. The blends were developed assuming two different RAP contents of 25 percent and 55 percent. This research generally concluded that if 100 percent mixing were to occur, at both the 25 and 55 percent replacement percentages the grade for the combined binder is one grade stiffer than the grade for the virgin AC, i.e. if the virgin AC was a PG 58-28 then the combined binder grade is PG 64-22. [West, Willis, Marasteanu, 2013]

Proper asphalt cement grade selection method for the City's mixes incorporating high percentage of RAP would have to be developed.

4.2 Asphalt Cement Containing Engine Oil Residue

The current City of Hamilton specifications for asphalt mixtures requires that the asphalt cement to be used in an asphalt mix should not include any Engine Oil Residue (EOR) and that samples of the AC will be obtained to carry out testing to determine whether EOR is present. This was included in the specifications in light of initial research undertaken at Queens University that claimed that the presence of EOR in the AC resulted in premature aging of the AC which in turn caused mix to perform poorly when placed in the pavement. However, more recent research provides other results. Researchers in the USA claim that there is no detrimental impact to the mix performance due to use of AC containing EOR. The asphalt industry generally remains somewhat undecided regarding the consequence of the inclusion of the EOR in AC and how to approach the subject.

Based on our current state of knowledge regarding the impact of AC containing EOR on mix performance, we recommend that the City specifications remove the requirement of not using AC containing EOR as well as the testing requirement. The current OPSS 1101 “Material Specification for Performance Graded Asphalt Cement” includes was amended from the May 2013 version to include an additional requirement for Ash Content Test. The purpose of the test it to identify deleterious material or filler in the AC. It is understood as that the determination of ash content is being used by the Ministry of Transportation of Ontario (MTO) to address the issue of EOR in AC prior to conclusive evidence regrading whether the inclusion of EOR results in premature ageing of the binder. It is our recommendation that the City specifications also reflect the requirement for testing and reporting of ash content of asphalt cement used in City mixes.

4.3 Preventive Maintenance of Existing Asphalt Surface Pavements

Pavement preservation treatments are increasingly being recognized for their potential to increase the service life of a pavement in an economically feasible manner. Currently road agencies are facing significant budget constraints and ageing pavement networks. Preventive maintenance treatments are hypothesized to provide road agencies with a cost effective treatment alternative that allows them to reduce the proportion of their road network requiring routine rehabilitation. In addition to the economical benefits provided to the road agencies, preventive maintenance treatments when applied correctly can also increase the overall level of service that the network is maintained at.

Preventive treatments are applied to pavements that are still in relatively good condition and have not reached or fallen below a level of service which would warrant rehabilitation. The candidate pavement should be in good structural condition. Preventive treatments are not applied to address structural or load bearing issues with the pavement but rather to address surface distresses to prevent further deterioration of the pavement. The principle behind pavement preservation is to apply a treatment earlier in the life of the pavement when the cost to return it to a required level of condition is significantly lower than the cost to apply major rehabilitation later in the pavement life.

Figure 1 shows the difference in timing in terms of pavement condition when a preventive treatment, rehabilitation and reconstruction are applied.

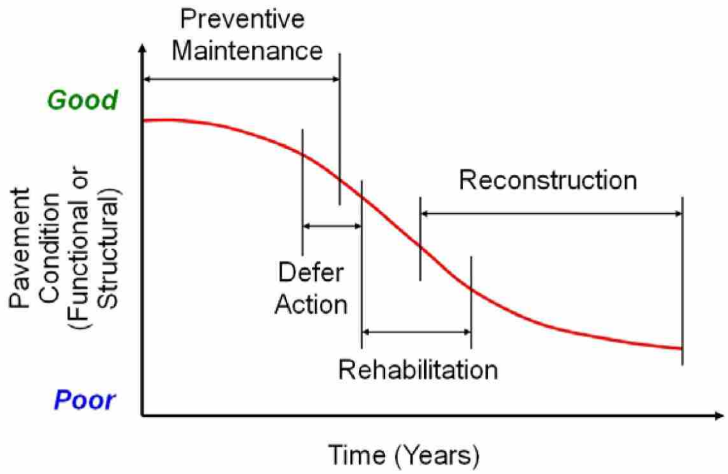

Figure 21: Timing of Preventive Maintenance, Rehabilitation and Reconstruction [Southern Slurry and Micro Surfacing Inc., 2012]

Figure 2 shows a theoretical representation of the cost savings that can be achieved by applying a timely preservation treatment as compared to applying reactive rehabilitation treatments later in the life of the pavement. it is also important to notice that the pavement condition is maintained at a high level.

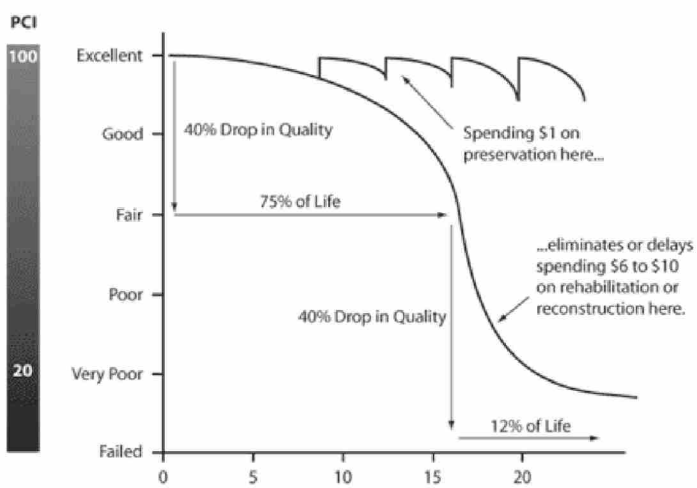

Figure 22: Theoretical Economical Benefits of Preventive Maintenance over Rehabilitation [Galehouse, Moulthorp, Hicks, 2006]

There is significant amount of literature available discussing the benefits of pavement preservation and yet other discussing the disadvantages and unknown aspects of preventive maintenance. Preventive treatments have been defined by the Federal Highway Administration (FHWA) as a "program strategy intended to arrest light deterioration, retard progressive failures, and reduce the need for routine maintenance and service activities" [NCHRP, 1989]. A critical factor that determines if a treatment will be successful is the time of application of the treatment on pavement sections and that those sections are well suited for a particular treatment.

In Ontario there are four basic approaches that are taken to pavement preservation. The approaches range from complete avoidance of treatments, due to poor past performances and lack of experience, through sporadic preventative treatment applications to frequent applications and a planned pavement preservation program. One of the biggest hurdles in the active usage of planned pavement preservation is the shift in mentality from "treating the worst first" to "treating good first". The key advantage of a pavement preservation program to municipalities is the increased certainty in the required maintenance budgets by reducing the proportion of necessary reactive treatments and steadily increasing the proportion of proactive treatments.

A 2010 report by the FHWA presented the estimated increase in pavement usable service life due to timely application of select preventive maintenance treatments. The study found that slurry sealing and chip sealing resulted in pavement service life increases of 4 to 7 years and 3 to 8 years, respectively. The least increase in pavement service life was found to be due to crack sealing which was estimated at approximately 2 to 4 years. For thin asphalt overlays, a very large range in pavement service life increases was reported from 3 to 23 years. [Zheng, Simpson, Hicks, 2010]

Examples of treatments that can be considered as preventive maintenance include crack sealing, patching, heater scarification, thin asphalt overlays, resurfacing for functional purposes, bonded concrete overlay, slurry sealing, seal coat, and microsurfacing. Some of these treatments are more commonly used than others.

4.3.1 Available Treatments

Several preventative maintenance treatments exist. Using the treatment in the appropriate application is very critical to the success of the project. The following is a list of current preventative maintenance treatments. The ones that are deemed suitable for the City's consideration are described further in the following sections.

- Crack sealing;
- Crack filling;
- Fog seals;
- Rejuvenating seals;
- Slurry seals;
- Chip seals;
- Cape seals;
- Sand seals;
- Microsurfacing;
- Bonded wearing course;
- Non-structural HMA overlay;
- Surface milling and non-structural overlay;
- Cold in-place surface recycling; and

- Hot in-place recycling.

4.3.1.1 Crack Sealing

Crack sealing of asphalt pavements is a treatment that is routinely undertaken by some municipalities and road authorities. Crack sealing or filling is carried out once cracks appear in the pavement surface with the purpose being to prevent the ingress of water into the pavement structure. Cracking sealing also prevents any debris or incompressibles getting lodged in the cracks which could results in further crack deterioration (crack spalling).

Asphalt crack sealing generally targets the working cracks. It involves routing or sawing a reservoir, preparing the reservoir through abrasive blasting, and thoroughly cleaning it with compressed air. Hot-poured, rubberized asphalt sealants are most commonly used to seal the reservoir. Asphalt crack filling is used for treating of non-working cracks. It includes cleaning the cracks with dried, compressed air and filling it with asphaltic material.

4.3.1.2 Slurry Seal

A slurry seal is a mixture of well graded fine aggregate, slow setting asphalt emulsion, water and mineral filler (most often Portland cement). Slurry seal incorporating polymer modified emulsion have longer life than conventional ones. Depending on the weather conditions (temperature, humidity and prevailing winds) the emulsion in a slurry seal can take from anywhere between two to eight hours to cure. The construction of slurry seals is covered by OPSS 337 "Construction Specification for Slurry Seal", dated November 2008.

During the slurry seal process aggregate, water, filler and asphalt emulsion are proportioned and blended together in a mixer and applied immediately to the pavement surface with a spreader box. The slurry is applied essentially one aggregate layer thick. Due to the fact that the asphalt emulsion in the slurry seal has to break and sure, this treatment should not be applied during rainy weather and the ambient temperature should be above about 10ºC.

Slurry seals will not perform well if the underlying pavement exhibits medium to major fatigue, linear or block cracking, rutting, roughness or shoving. It should be applied only where the existing pavement is in good structural condition and the surface is stable with only low-severity cracking.

This treatment seals the existing pavement surface, slows surface raveling, seals small cracks and improves surface friction. The treatment is most effective where the primary problem is excessive oxidation and hardening of the asphalt concrete or where there are aggregate 'pop-outs' in the asphalt surface. Slurry seals do not have a strong skeleton and are typically applied as one aggregate layer thick. They are not suitable to correct surface irregularities and rutting.

The typical cost of slurry seals is about $2.50/m^2. Experienced slurry seal contractors are available throughout Ontario and the product is highly reliable. The life expectancy is up to 7 years.

Over the last several years another type of slurry seal, a cape seal treatment has been used in Ontario. In this process, a slurry seal is applied to a newly constructed chip seal to improve the retention of the stone chips and seal the open voids. The cost of a cape seal is about $4.00/m^2. This treatment has a life expectancy from 9 to 15 years with the typical life about 9 years before re-application.

4.3.1.3 Chip Seals

Chip seals, also known as surface treatments, consist of a single application of conventional, high float or polymer modified asphalt emulsion followed immediately by an aggregate cover. The goal is to have the aggregate particles embed themselves into the asphalt layer with about 30 percent of each particle exposed to provide texture. Application of two successive layers is referred to as a double chip seal. Chip seals can be applied at any time in a pavement's life as an economical, durable and widely available treatment. The construction of chip seal is covered by OPSS 304 "Construction Specification for Single and Double Surface Treatment", dated November 2006.

Chip seals can waterproof the pavement surface, provide sealing of low severity cracks, and restore surface friction. They also slow down the asphalt cement oxidation process within the original asphalt surface layer. Chip seals are not effective on pavements exhibiting medium to major fatigue, linear or block cracking, rutting, roughness and shoving.

The typical cost of a chip seal treatment in Ontario ranges from \$1.40 to \$1.75/m^2. The unit cost for the chip seal is higher if a polymer modified asphalt emulsion is used rather than a conventional emulsion. Numerous qualified and experienced contractors are available throughout Ontario.

Chip seals should only be applied in favourable weather conditions and their performance can be adversely affected if installed in wet weather. As with the slurry seals, the asphalt emulsion used in chip seals needs time to break and then cure. Due to the risk of loose chips and excessive noise, chip seals are now typically only used on low volume rural roads in Ontario.

4.3.1.4 Microsurfacing

Microsurfacing is a mixture of polymer modified emulsion, well graded, high quality crushed mineral aggregate (typically 9.5 mm minus), mineral filler (normally Portland cement), water and chemical additives that control the break time, i.e. the time for the emulsion to achieve a set. The aggregates used in microsurfacings have superior physical property requirements in terms of strength, durability and polishing characteristics. Microsurfacing is a chemically controlled process. The materials are mixed in a truck mounted traveling plant and then deposited into a spreader box. No compaction is needed and traffic may be allowed on the mat within an hour after placement. The construction of a microsurfacing is covered by OPSS 336 "Construction Specification for Micro-Surfacing", dated November 2009. This pavement treatment may involve two coats including a scratch or leveling coat followed by a surface coat. A typical rate of application that can be achieved by an experienced contractor is about 10 lane kilometers per day.

Microsurfacing is applied on streets or highways carrying medium to high volume traffic, on high speed roads and airfield pavements. The pavements should be in good structural condition and not exhibiting any significant structural surface distresses. The treatment should not to be used on pavements with moderate to heavy alligator cracking. Microsurfacing has a strong skeleton and can be applied in thin and also relatively thick layers; it is very effective in correcting surface irregularities including minor transverse profile corrections, and low to medium severity wheel track rutting.

Microsurfacing provides a high quality skid resistant surface for an existing asphalt concrete pavement, seals the pavement surface, restores surface profile, eliminates hydroplaning, and provides a surface that is more resistant to rutting and shoving. Microsurfacing is applied at ambient temperatures and has low energy requirements. Due to its quick application rate, it causes minimum disruption to traffic. The typical cost of the treatment ranges from \$3.75 to \$4.00/m^2 and there are a number qualified microsurfacing contractors in Ontario. Life expectancy of microsurfacing is about 7 years; however, there are cases in Ontario where its life was 10 and more years.

4.3.1.5 Bonded Wearing Course

For a bonded wearing course (previously known as Nova Chip) a layer of heavy polymer modified emulsion is applied to the road surface, and within seconds, a very thin layer of gap graded HMA is placed on the emulsion using a screed. The water driven from the emulsion cools the HMA, setting both materials and providing a bond to the underlying surface. This treatment utilizes very high quality aggregates, that must be 100 percent crushed and cubical (trap rock is typically used). The typical layer thickness for a bonded wearing course is about 12.5 mm but it can be increased up to 40 mm. Following placement of the mat with the screed, it is seated with a static roller. The rate of placement can be up to two to three times faster than the rate of conventional HMA paving. Once the process has been completed, the road can be opened to traffic as soon as the mix has cooled down to ambient temperatures.

The bonded wearing course provides a very flexible surface layer. It is an open graded material, with high air void content and is bound with soft polymer modified asphalt cement. It has been applied successfully on roads carrying medium to very heavy traffic (up to 100,000 vehicles per day). As with the other preventive maintenance treatments, a bonded wearing course is not intended to improve the structural capacity of the road; however, it provides a flexible thin surfacing that greatly enhances ride quality.

This treatment provides very good frictional characteristics, seals the pavement surface, stops surface distresses, and reduces hydroplaning. Additionally, a bonded wearing course is generally considered to be quieter than a conventional HMA surface. It also provides excellent bond to the existing pavement surface and durability. The life of this treatment can range anywhere between 7 to 10 years and the typical cost ranges from \$4.50 to \$5.00/m^2.

4.3.2 Recommendations for Application of Preventive Maintenance

Golder will be pleased to carry out Staff training on the application of pavement preventive treatment. This would also include the basis of hot-in-place recycling. It is of particular interest to investigate the application of some of the preventive treatment methods to pavements incorporating steel slag.

The City has a large number of roads with steel slag mixes paved 20 or more years ago. The majority of those pavements are in good structural condition; however, they exhibit extensive surface cracking and raveling. The application of slurry seal or microsurfacing may be considered as a much lower cost solution to mill and overlay. We recommend that addition of fibre to the slurry seal or microsurfacing mixes should be considered. There is not a practical example of the application of both treatments on old steel slag pavements. Therefore, Golder recommends a trial strip be placed. Golder would be pleased to provide the specifications and develop the procedure for the trial strip.

5.0 CLOSURE

We trust that this report meets your present requirements. If you have any questions or require additional information please do not hesitate to contact this office.

Report Signature Page

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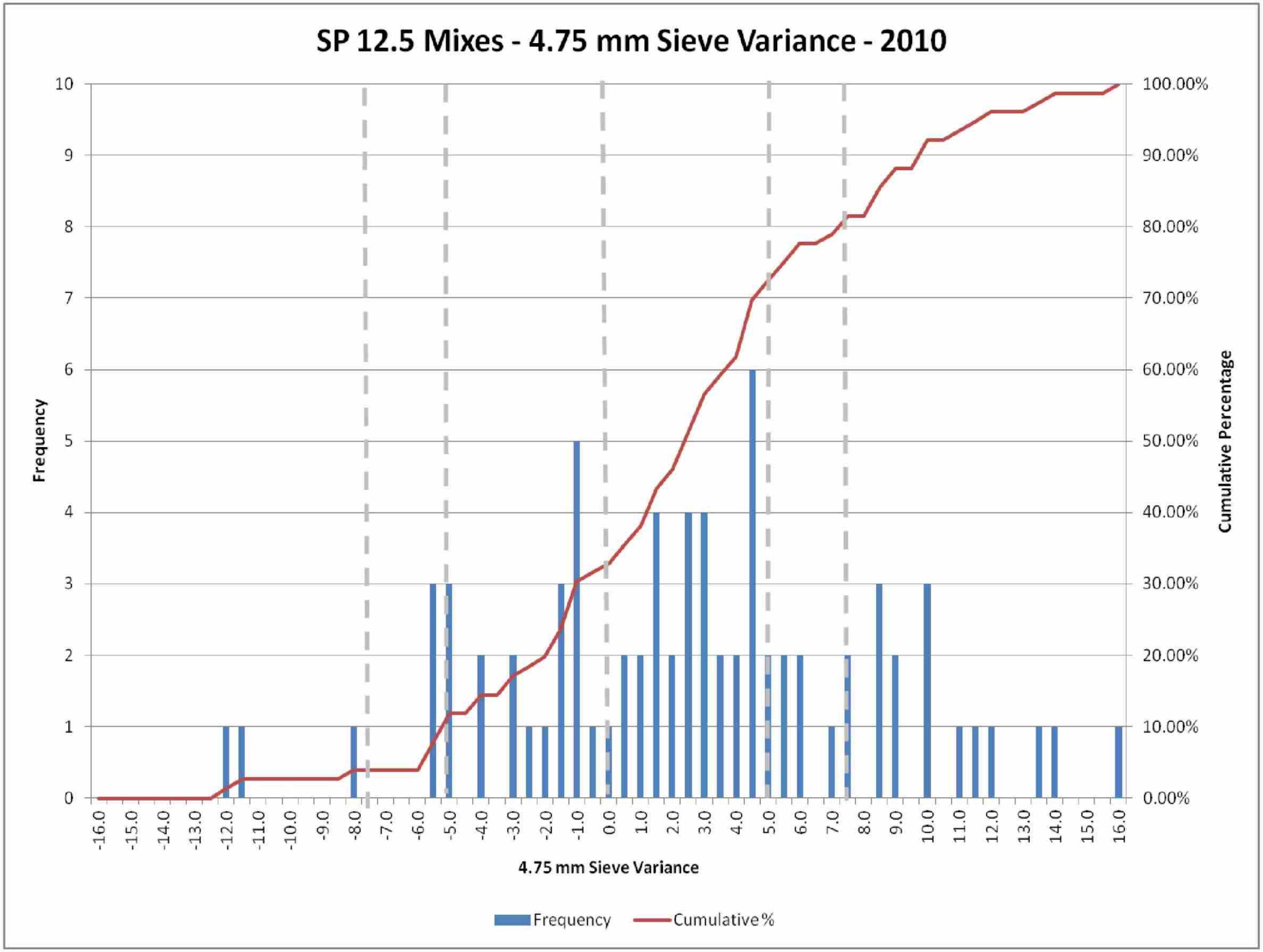
APPENDIX A

HMA Testing Results Distribution in 2010, 2011, 2012 and 2013

2010



SP 12.5 Mixes - 4.75 mm Sieve Variance - 2010
Frequency
Cumulative Percentage
4.75 mm Sieve Variance
0
1
2
3
4
5
6
7
8
9
10
0.00%
10.00%
20.00%
30.00%
40.00%
50.00%
60.00%
70.00%
80.00%
90.00%
100.00%
-16.0
-15.0
-14.0
-13.0
-12.0
-11.0
-10.0
-9.0
-8.0
-7.0
-6.0
-5.0
-4.0
-3.0
-2.0
-1.0
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
10.0
11.0
12.0
13.0
14.0
15.0
16.0
Frequency
Cumulative %

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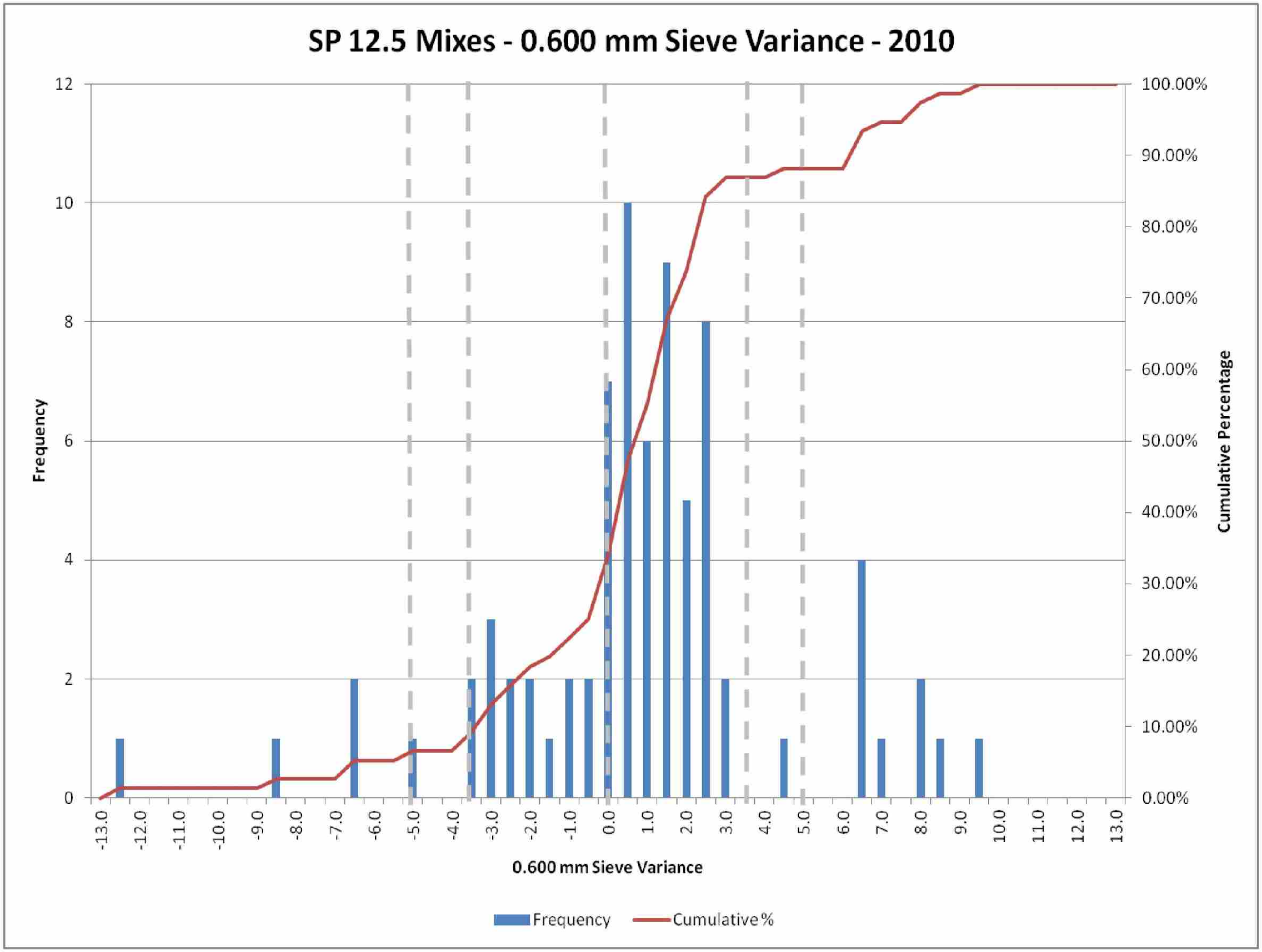
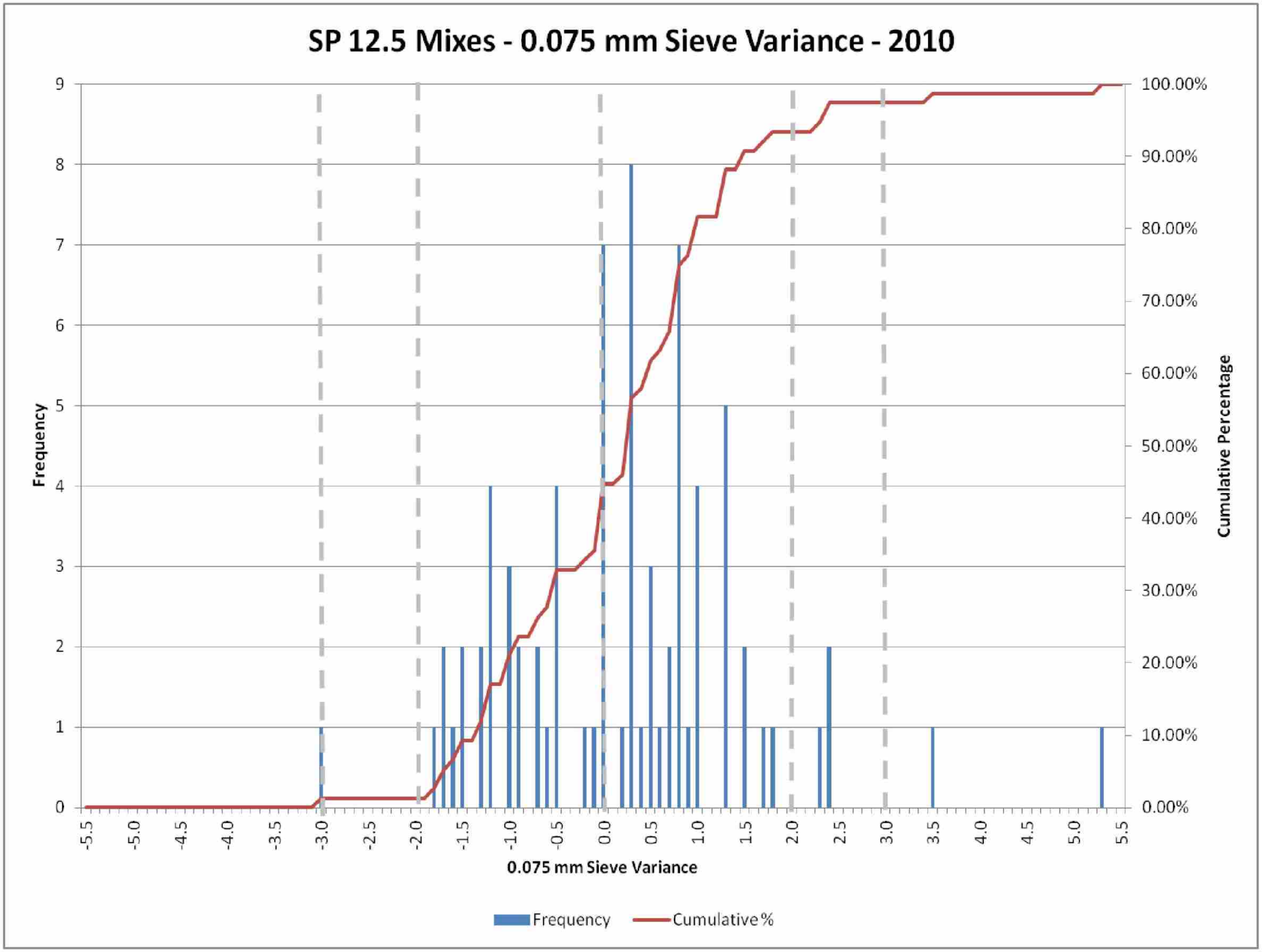
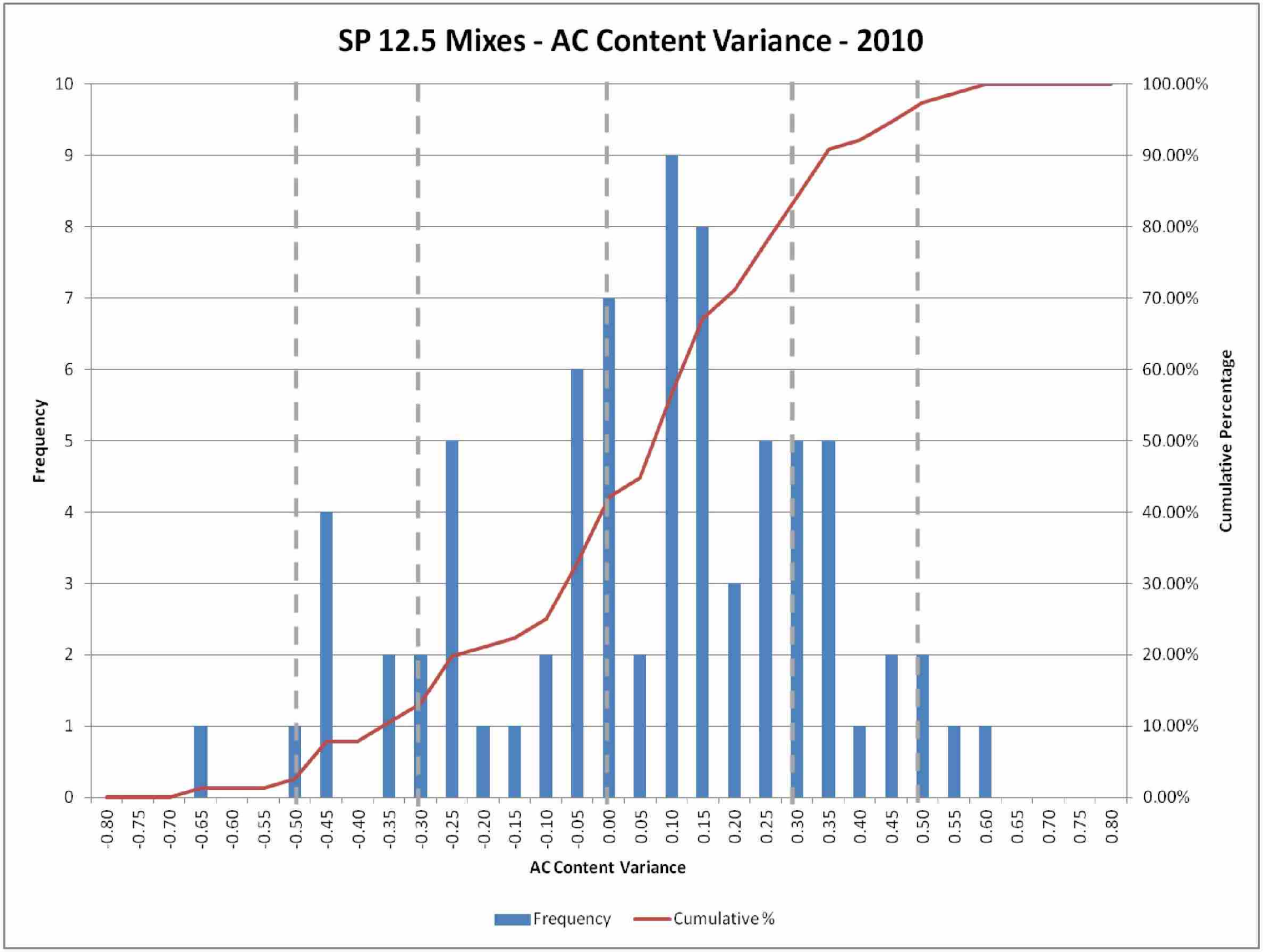


SP 12.5 Mixes - 0.600 mm Sieve Variance - 2010
Frequency
Cumulative Percentage
0.600 mm Sieve Variance
Frequency
Cumulative %

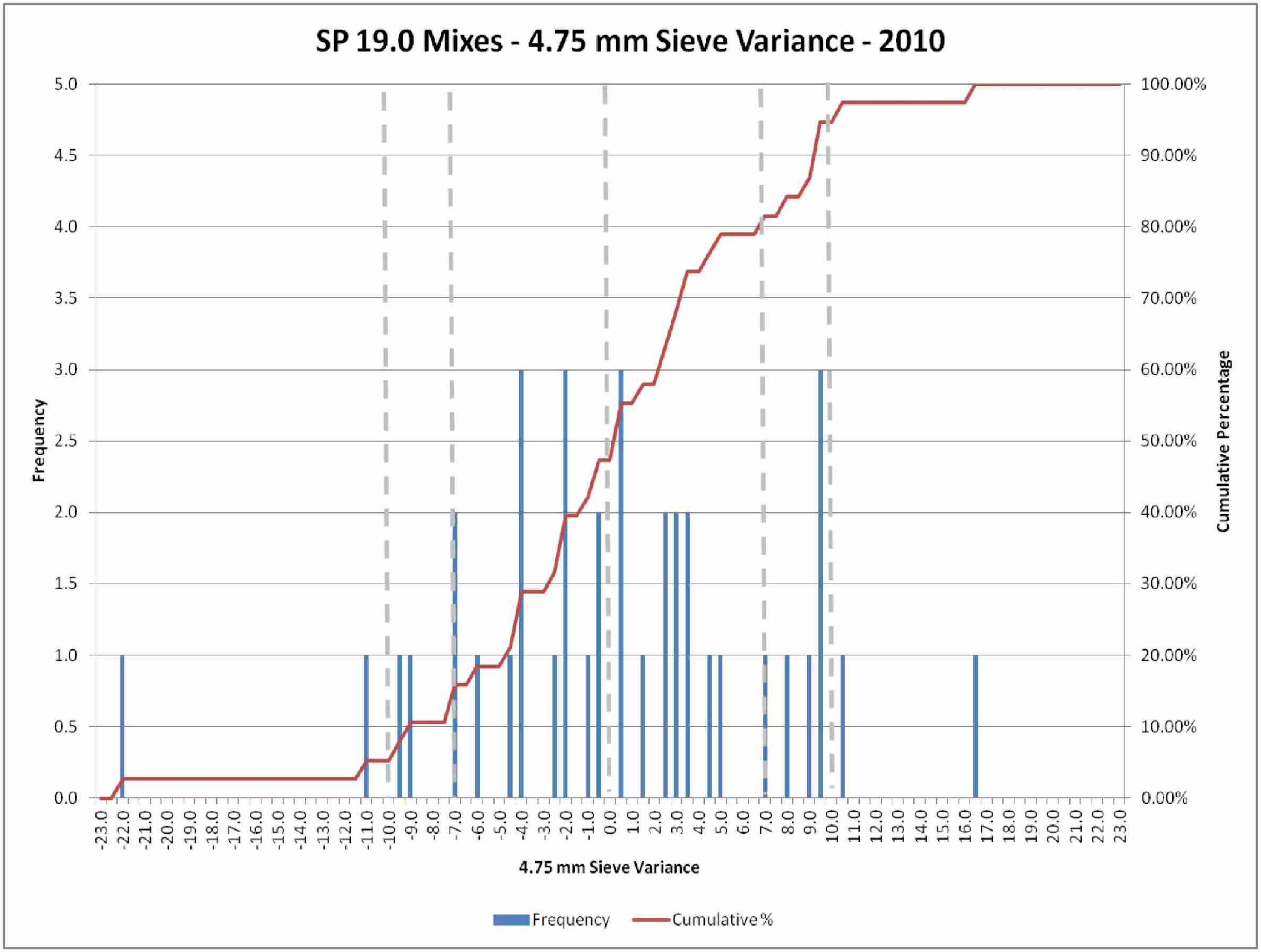


SP 12.5 Mixes - 0.075 mm Sieve Variance - 2010
Frequency
Cumulative Percentage
0.075 mm Sieve Variance
0
1
2
3
4
5
6
7
8
9
0.00%
10.00%
20.00%
30.00%
40.00%
50.00%
60.00%
70.00%
80.00%
90.00%
100.00%
-5.5
-5.0
-4.5
-4.0
-3.5
-3.0
-2.5
-2.0
-1.5
-1.0
-0.5
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0
5.5
Frequency
Cumulative %

GOL0007504

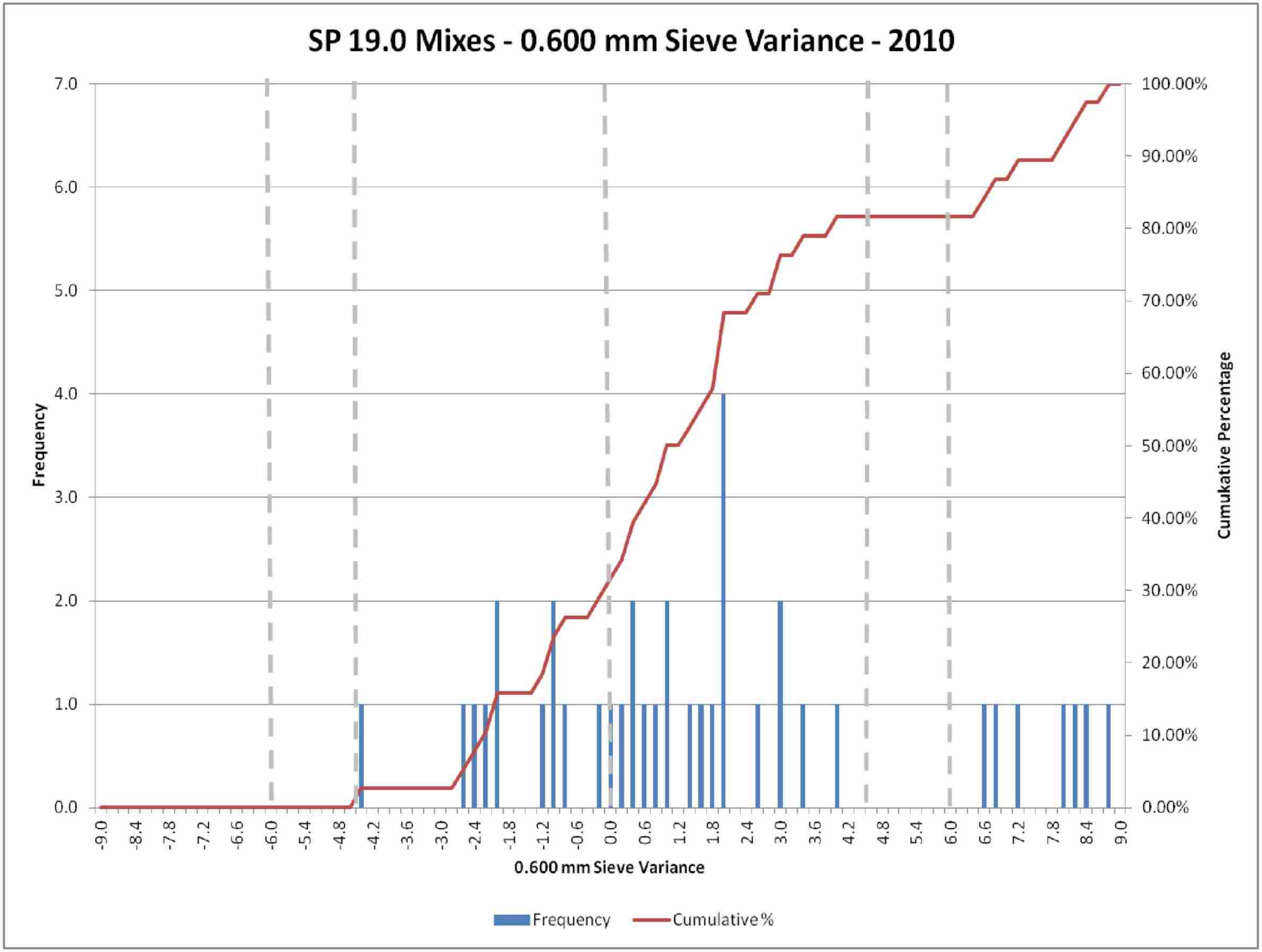

SP 12.5 Mixes - AC Content Variance - 2010
Frequency
Cumulative Percentage
AC Content Variance
0
1
2
3
4
5
6
7
8
9
10
0.00%
10.00%
20.00%
30.00%
40.00%
50.00%
60.00%
70.00%
80.00%
90.00%
100.00%
-0.80
-0.75
-0.70
-0.65
-0.60
-0.55
-0.50
-0.45
-0.40
-0.35
-0.30
-0.25
-0.20
-0.15
-0.10
-0.05
0.00
0.05
0.10
0.15
0.20
0.25
0.30
0.35
0.40
0.45
0.50
0.55
0.60
0.65
0.70
0.75
0.80
Frequency
Cumulative %

GOL0007504

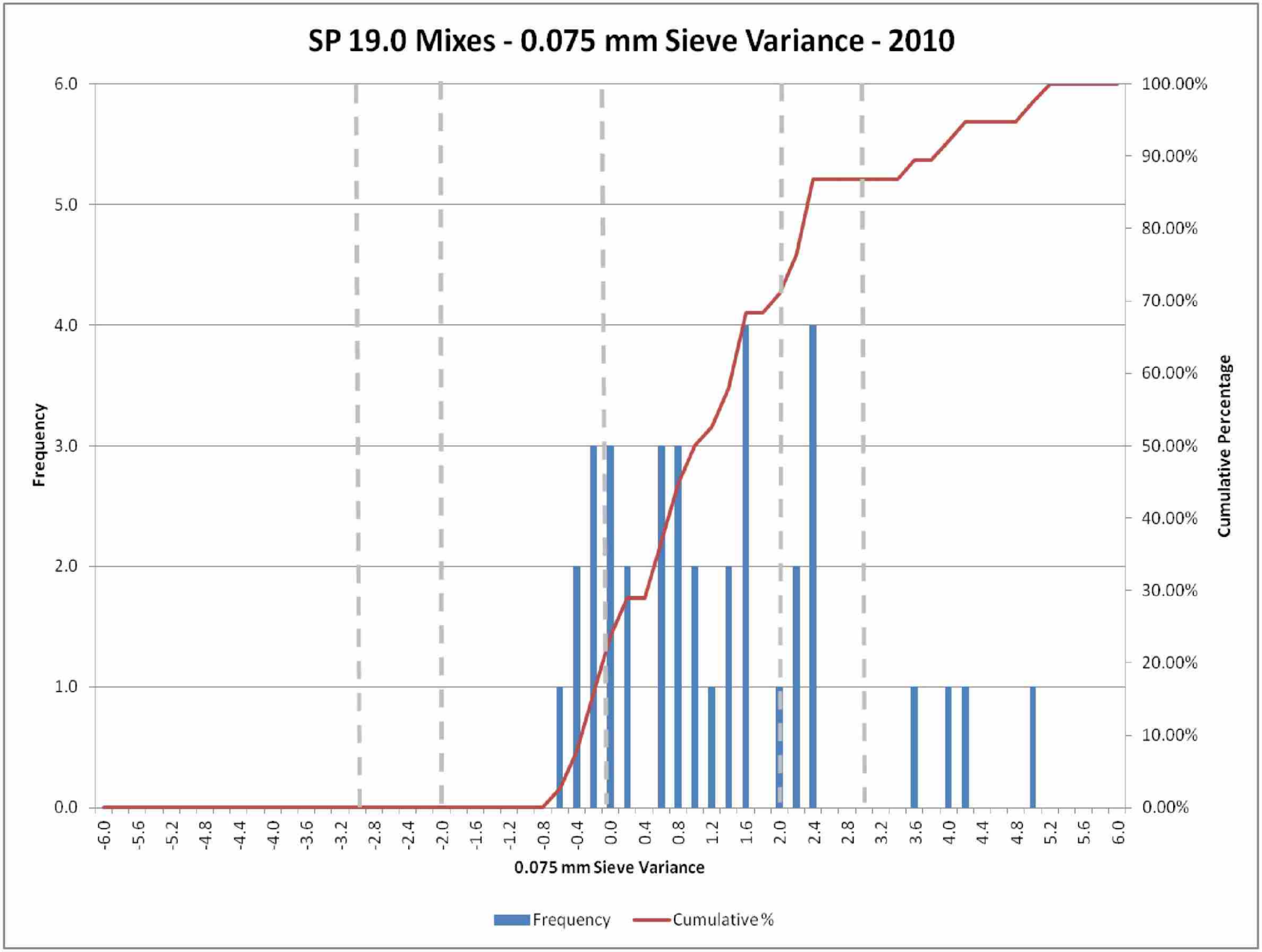


SP 19.0 Mixes - 4.75 mm Sieve Variance - 2010
Frequency
Cumulative Percentage
4.75 mm Sieve Variance
Frequency
Cumulative %

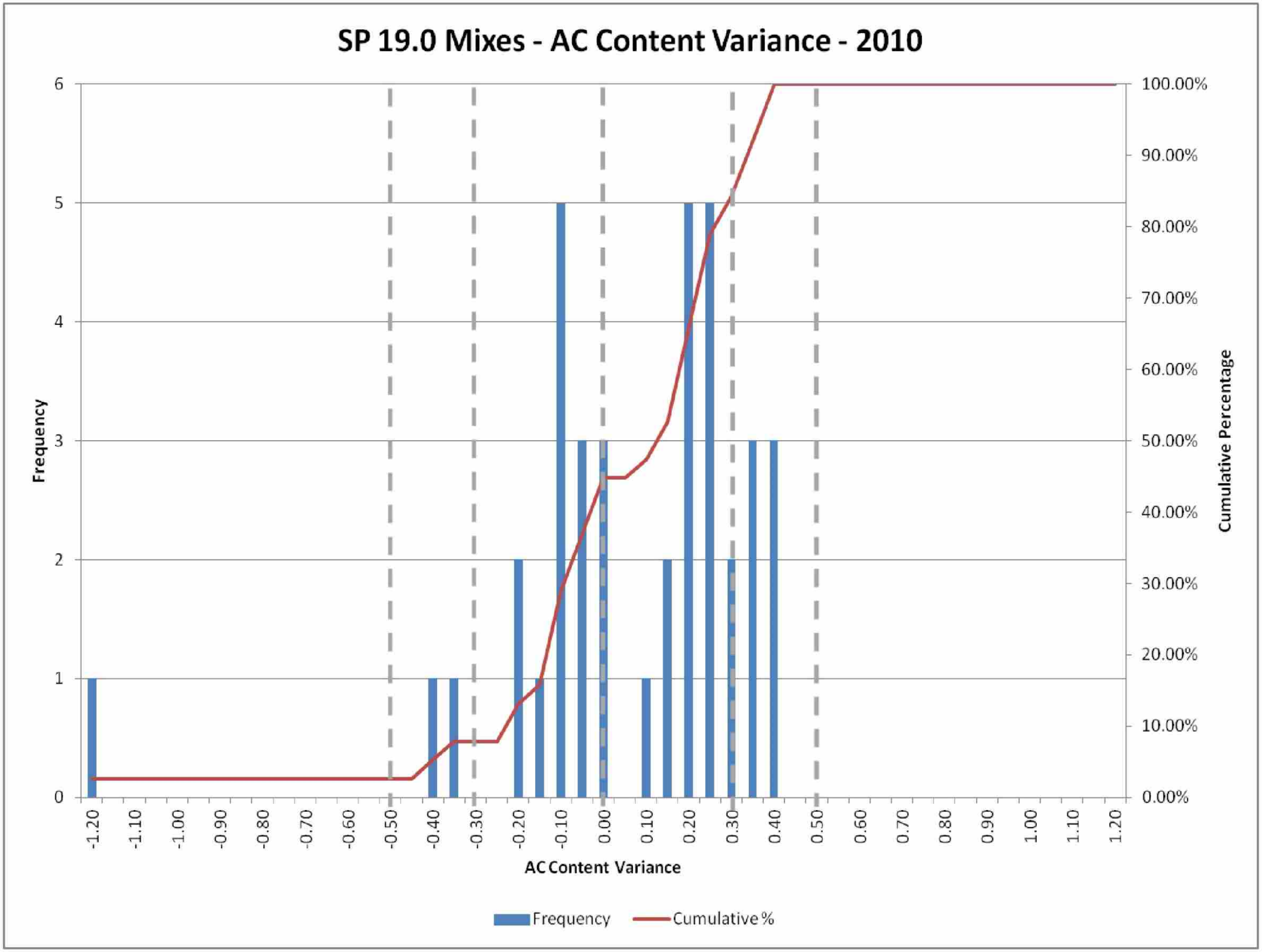
GOL0007504



SP 19.0 Mixes - 0.600 mm Sieve Variance - 2010
Frequency
Cumukative Percentage
0.600 mm Sieve Variance
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
100.00%
90.00%
80.00%
70.00%
60.00%
50.00%
40.00%
30.00%
20.00%
10.00%
0.00%
-9.0 -8.4 -7.8 -7.2 -6.6 -6.0 -5.4 -4.8 -4.2 -3.6 -3.0 -2.4 -1.8 -1.2 -0.6 0.0 0.6 1.2 1.8 2.4 3.0 3.6 4.2 4.8 5.4 6.0 6.6 7.2 7.8 8.4 9.0
Frequency
Cumulative %

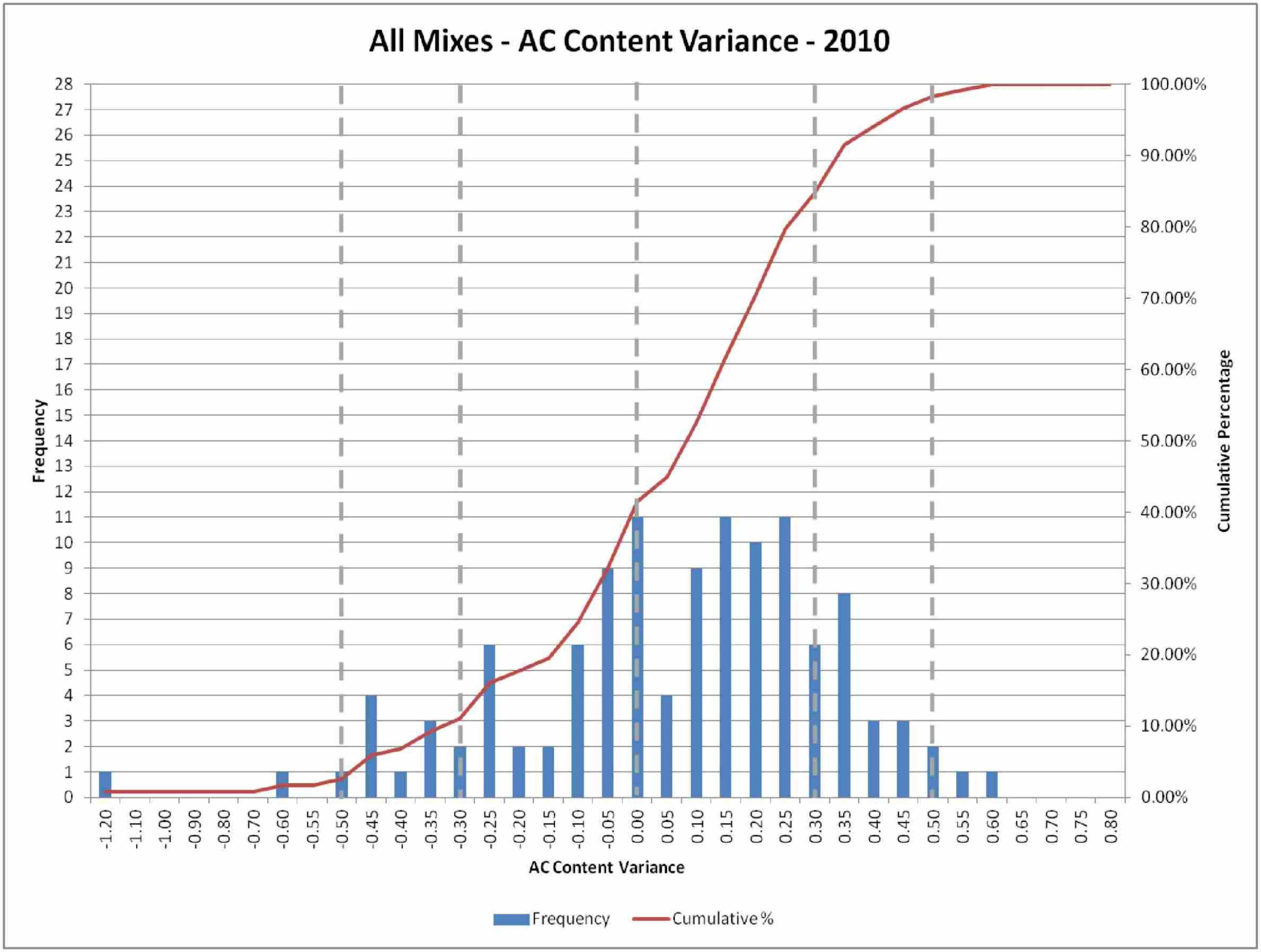


SP 19.0 Mixes - 0.075 mm Sieve Variance - 2010
Frequency
Cumulative Percentage
6.0
5.0
4.0
3.0
2.0
1.0
0.0
100.00%
90.00%
80.00%
70.00%
60.00%
50.00%
40.00%
30.00%
20.00%
10.00%
0.00%
-6.0 -5.6 -5.2 -4.8 -4.4 -4.0 -3.6 -3.2 -2.8 -2.4 -2.0 -1.6 -1.2 -0.8 -0.4 0.0 0.4 0.8 1.2 1.6 2.0 2.4 2.8 3.2 3.6 4.0 4.4 4.8 5.2 5.6 6.0
0.075 mm Sieve Variance
Frequency
Cumulative %

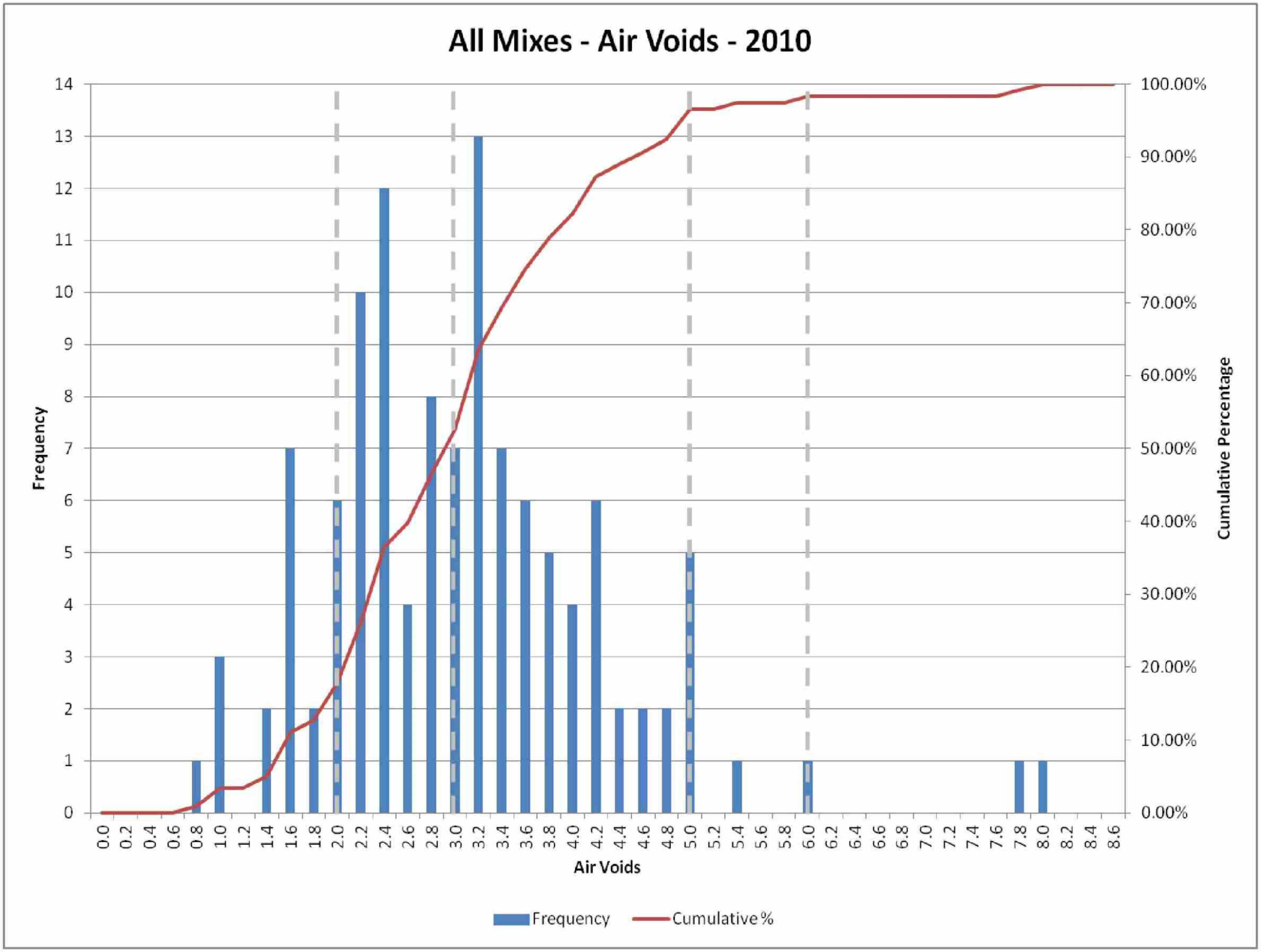


SP 19.0 Mixes - AC Content Variance - 2010
Frequency
Cumulative Percentage
AC Content Variance
0
1
2
3
4
5
6
0.00%
10.00%
20.00%
30.00%
40.00%
50.00%
60.00%
70.00%
80.00%
90.00%
100.00%
-1.20
-1.10
-1.00
-0.90
-0.80
-0.70
-0.60
-0.50
-0.40
-0.30
-0.20
-0.10
0.00
0.10
0.20
0.30
0.40
0.50
0.60
0.70
0.80
0.90
1.00
1.10
1.20
Frequency
Cumulative %

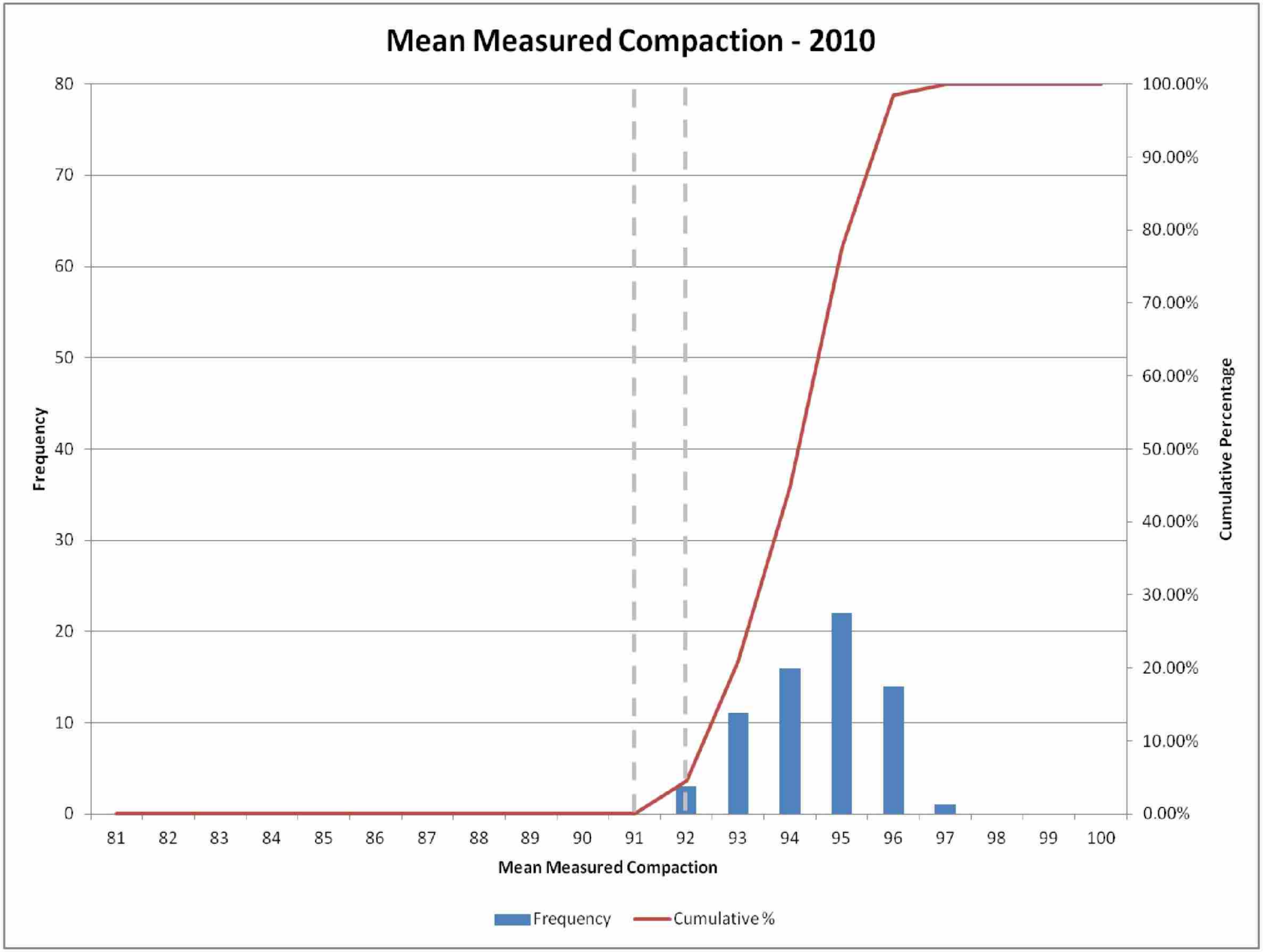
GOL0007504



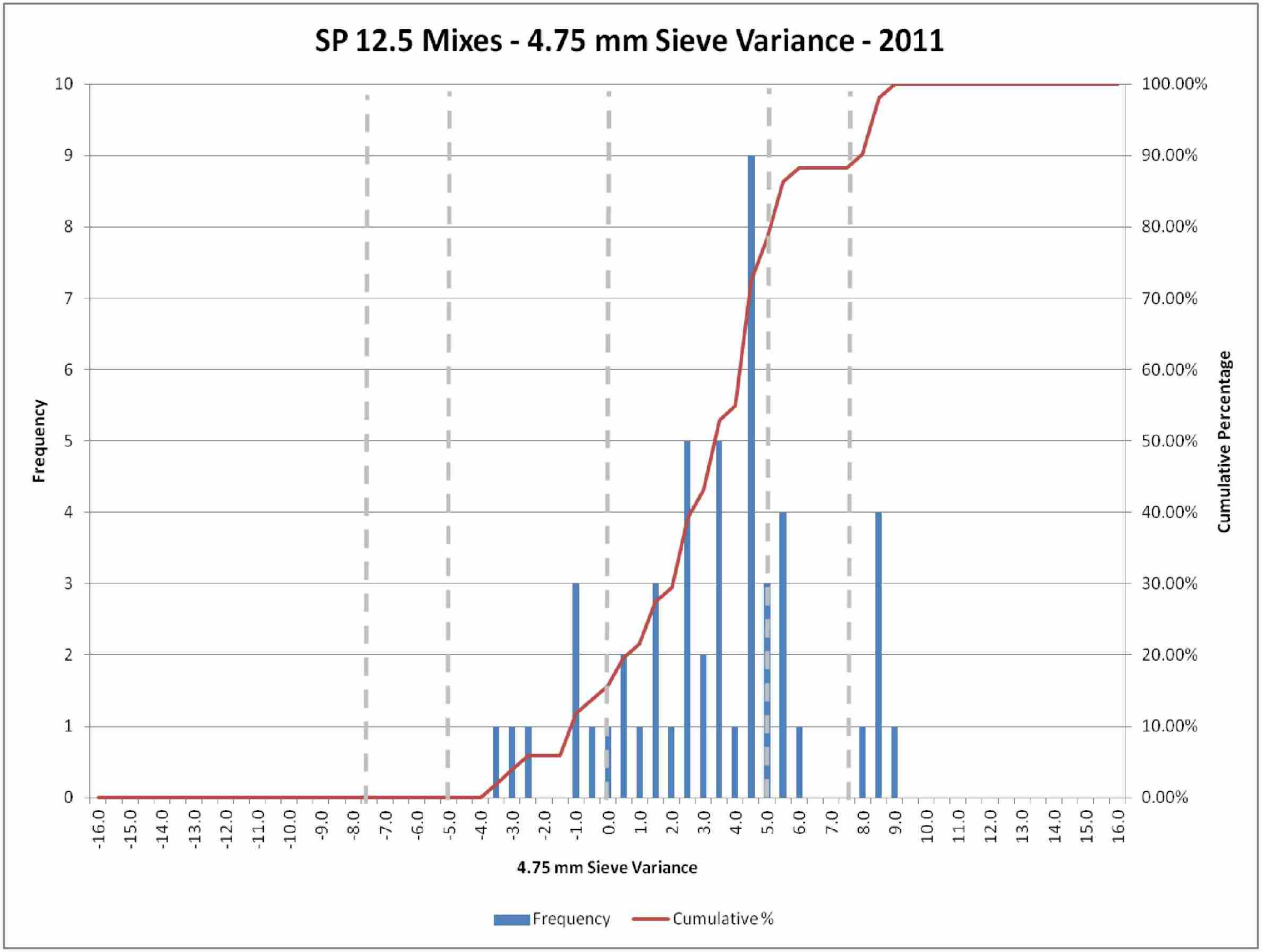
All Mixes - AC Content Variance - 2010
Frequency
Cumulative Percentage
AC Content Variance
28
27
26
25
24
23
22
21
20
19
18
17
16
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
0
100.00%
90.00%
80.00%
70.00%
60.00%
50.00%
40.00%
30.00%
20.00%
10.00%
0.00%
-1.20
-1.10
-1.00
-0.90
-0.80
-0.70
-0.60
-0.55
-0.50
-0.45
-0.40
-0.35
-0.30
-0.25
-0.20
-0.15
-0.10
-0.05
0.00
0.05
0.10
0.15
0.20
0.25
0.30
0.35
0.40
0.45
0.50
0.55
0.60
0.65
0.70
0.75
0.80
Frequency
Cumulative %

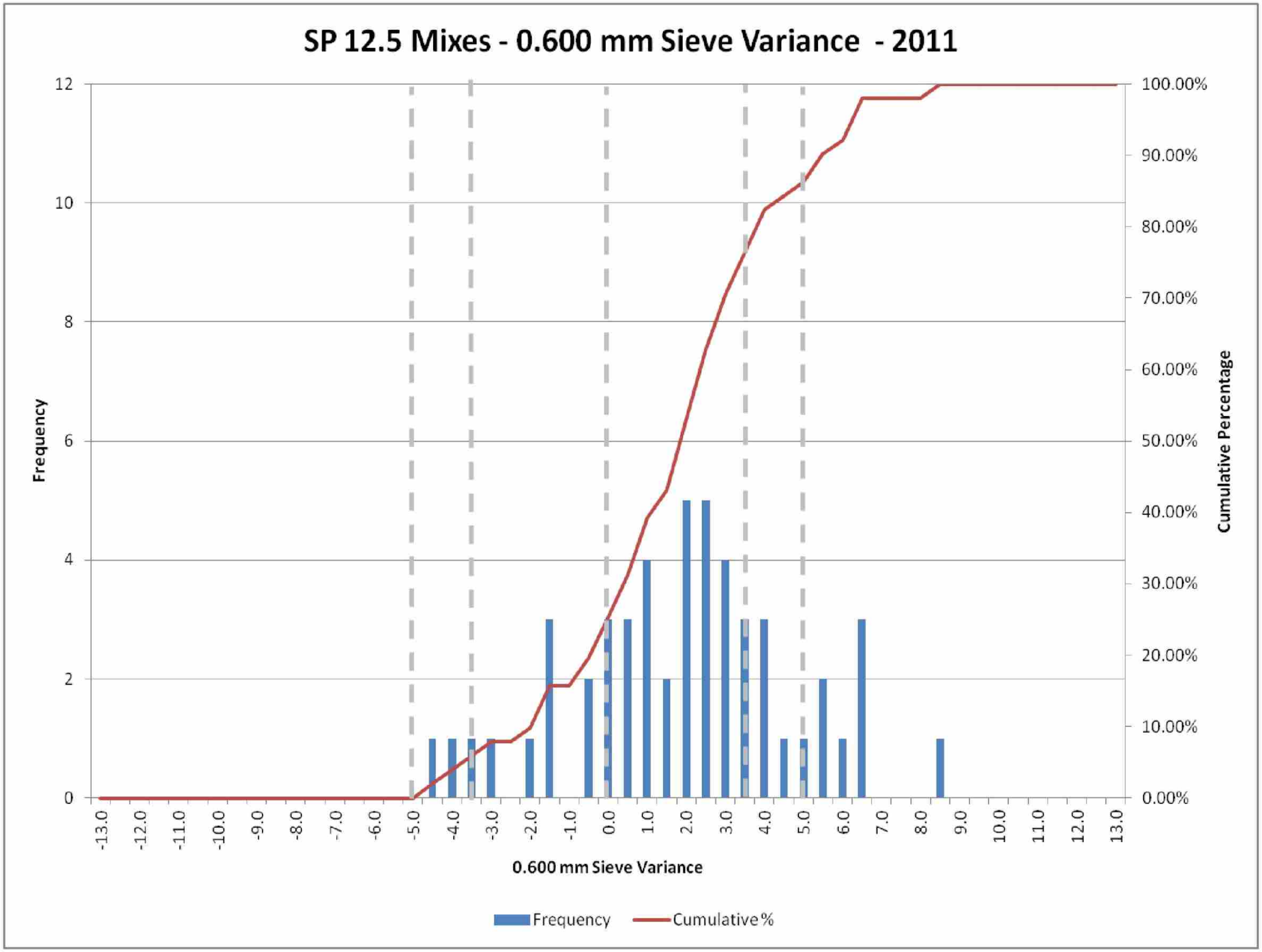

All Mixes - Air Voids - 2010
Frequency
Cumulative Percentage
Air Voids
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14
0.00% 10.00% 20.00% 30.00% 40.00% 50.00% 60.00% 70.00% 80.00% 90.00% 100.00%
0.0 0.2 0.4 0.6 0.8 1.0 1.2 1.4 1.6 1.8 2.0 2.2 2.4 2.6 2.8 3.0 3.2 3.4 3.6 3.8 4.0 4.2 4.4 4.6 4.8 5.0 5.2 5.4 5.6 5.8 6.0 6.2 6.4 6.6 6.8 7.0 7.2 7.4 7.6 7.8 8.0 8.2 8.4 8.6
Frequency
Cumulative %

GOL0007504

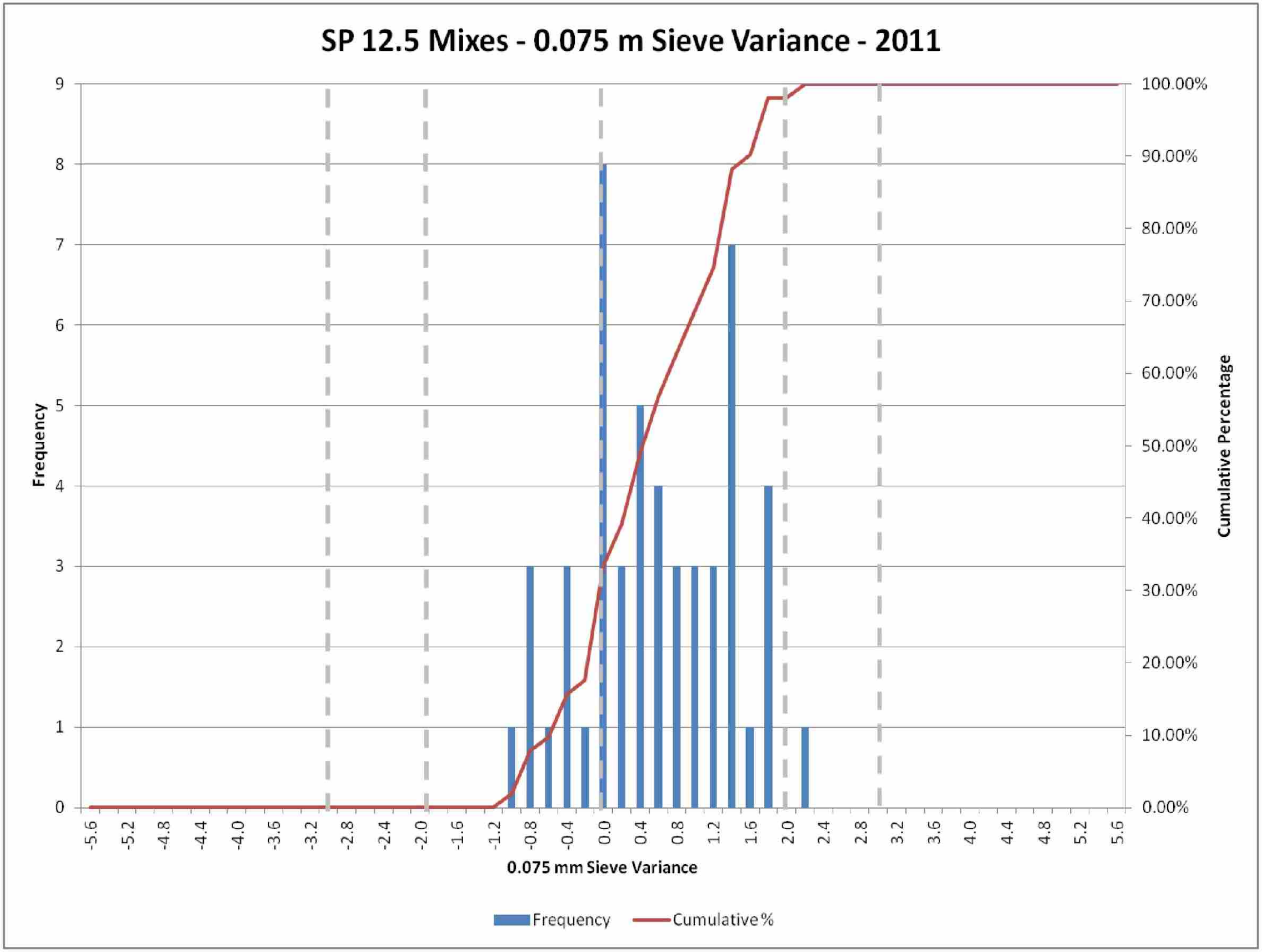

Mean Measured Compaction - 2010
80
70
60
50
40
30
20
10
0
Frequency
100.00%
90.00%
80.00%
70.00%
60.00%
50.00%
40.00%
30.00%
20.00%
10.00%
0.00%
Cumulative Percentage
81 82 83 84 85 86 87 88 89 90 91 92 93 94 95 96 97 98 99 100
Mean Measured Compaction
Frequency
Cumulative %

2011


SP 12.5 Mixes - 4.75 mm Sieve Variance - 2011
Frequency
Cumulative Percentage
4.75 mm Sieve Variance
Frequency
Cumulative %
0
1
2
3
4
5
6
7
8
9
10
0.00%
10.00%
20.00%
30.00%
40.00%
50.00%
60.00%
70.00%
80.00%
90.00%
100.00%
-16.0 -15.0 -14.0 -13.0 -12.0 -11.0 -10.0 -9.0 -8.0 -7.0 -6.0 -5.0 -4.0 -3.0 -2.0 -1.0 0.0 1.0 2.0 3.0 4.0 5.0 6.0 7.0 8.0 9.0 10.0 11.0 12.0 13.0 14.0 15.0 16.0

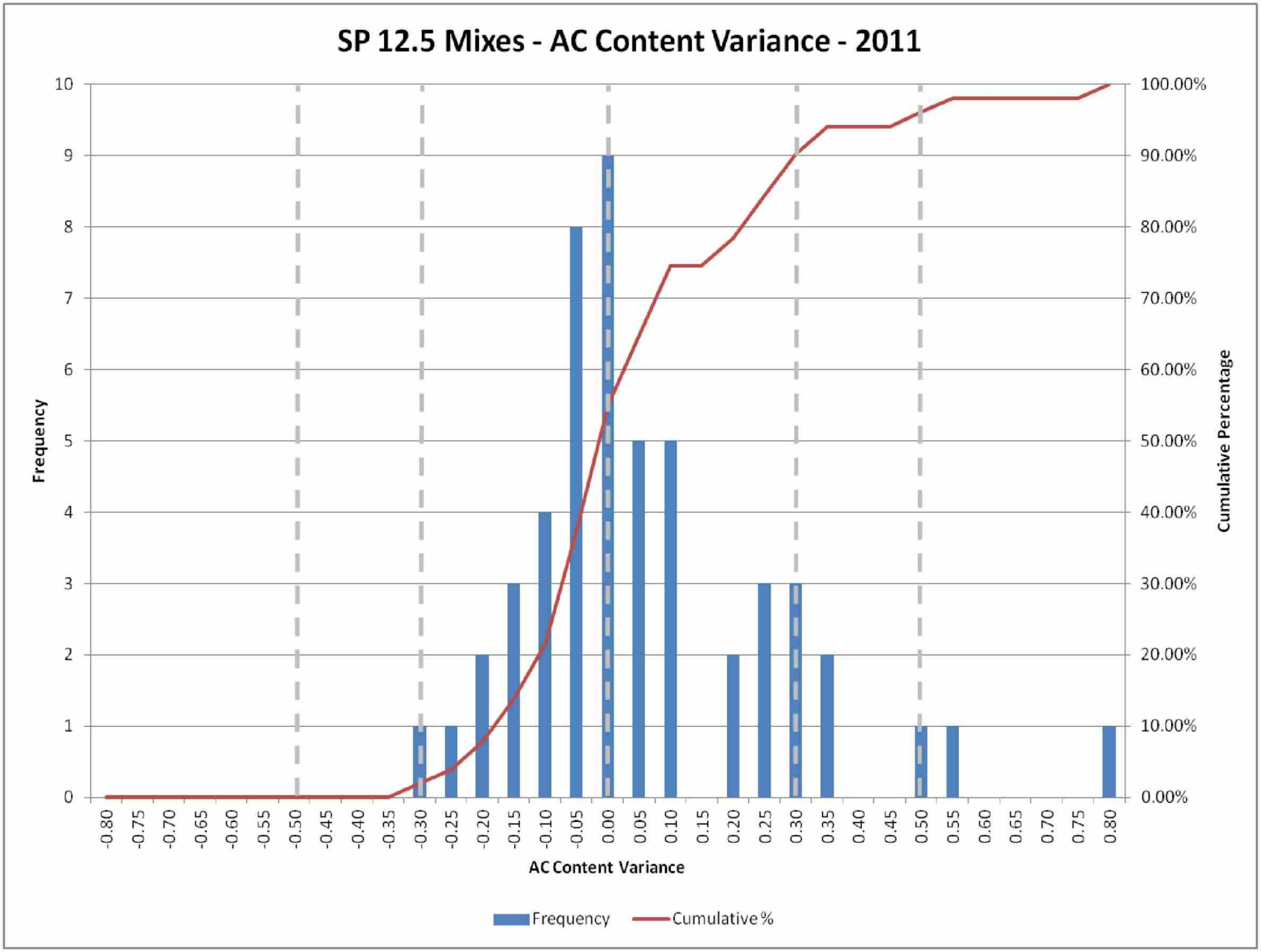


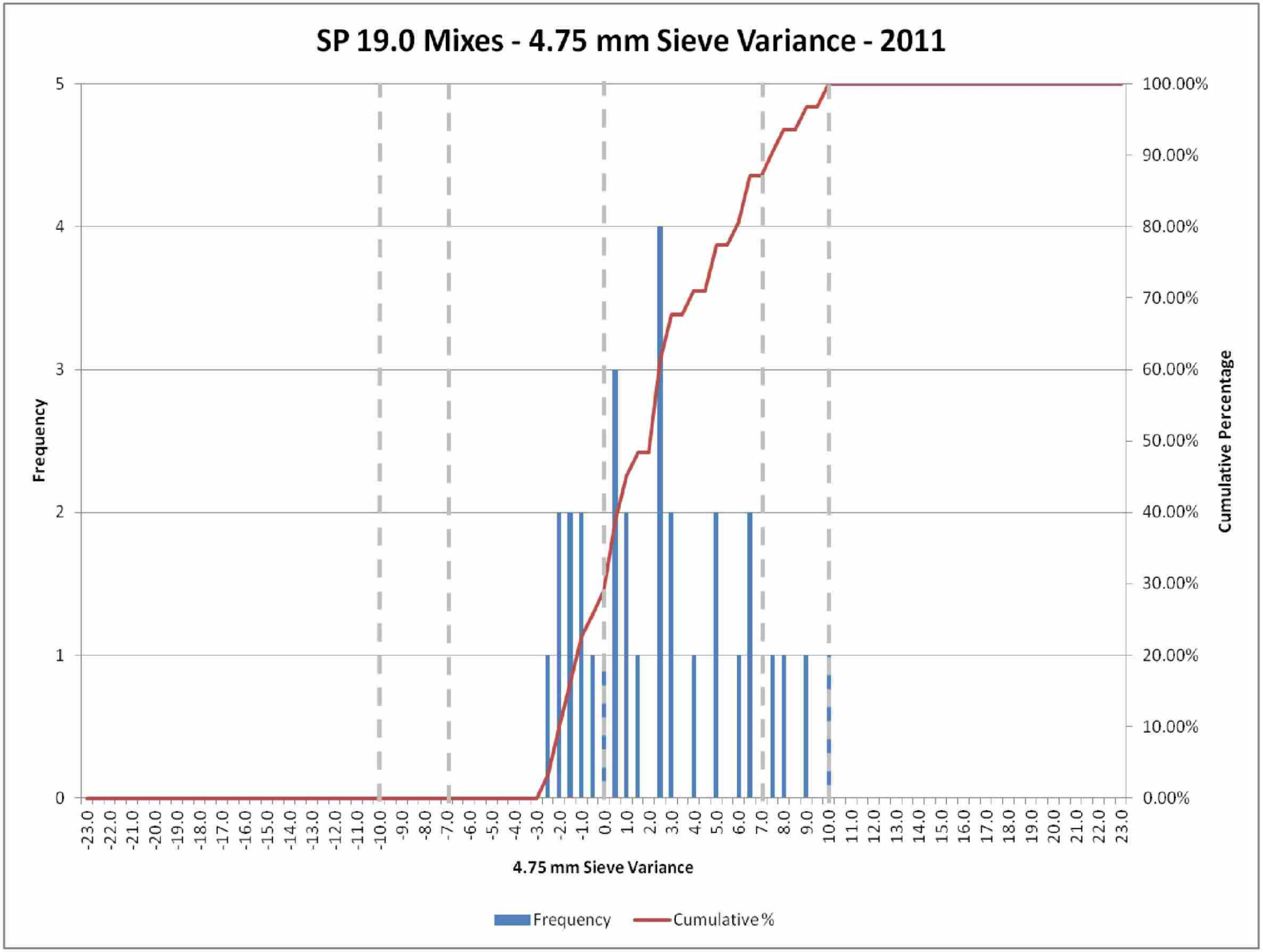
SP 12.5 Mixes - 0.600 mm Sieve Variance - 2011
Frequency
Cumulative Percentage
0.600 mm Sieve Variance
Frequency
Cumulative %

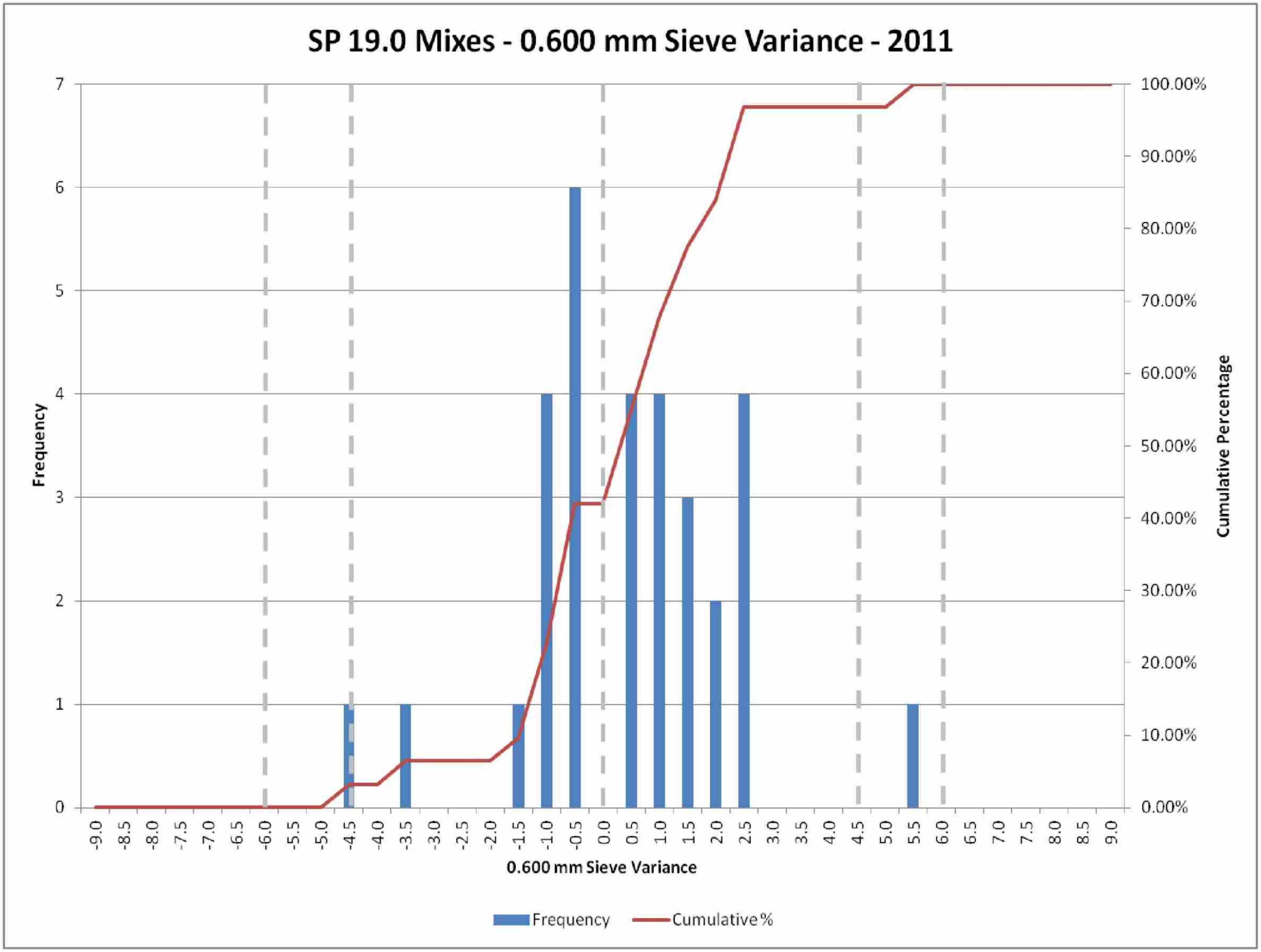


SP 12.5 Mixes - 0.075 m Sieve Variance - 2011
Frequency
Cumulative Percentage
0.075 mm Sieve Variance
0
1
2
3
4
5
6
7
8
9
0.00%
10.00%
20.00%
30.00%
40.00%
50.00%
60.00%
70.00%
80.00%
90.00%
100.00%
-5.6
-5.2
-4.8
-4.4
-4.0
-3.6
-3.2
-2.8
-2.4
-2.0
-1.6
-1.2
-0.8
-0.4
0.0
0.4
0.8
1.2
1.6
2.0
2.4
2.8
3.2
3.6
4.0
4.4
4.8
5.2
5.6
Frequency
Cumulative %

GOL0007504

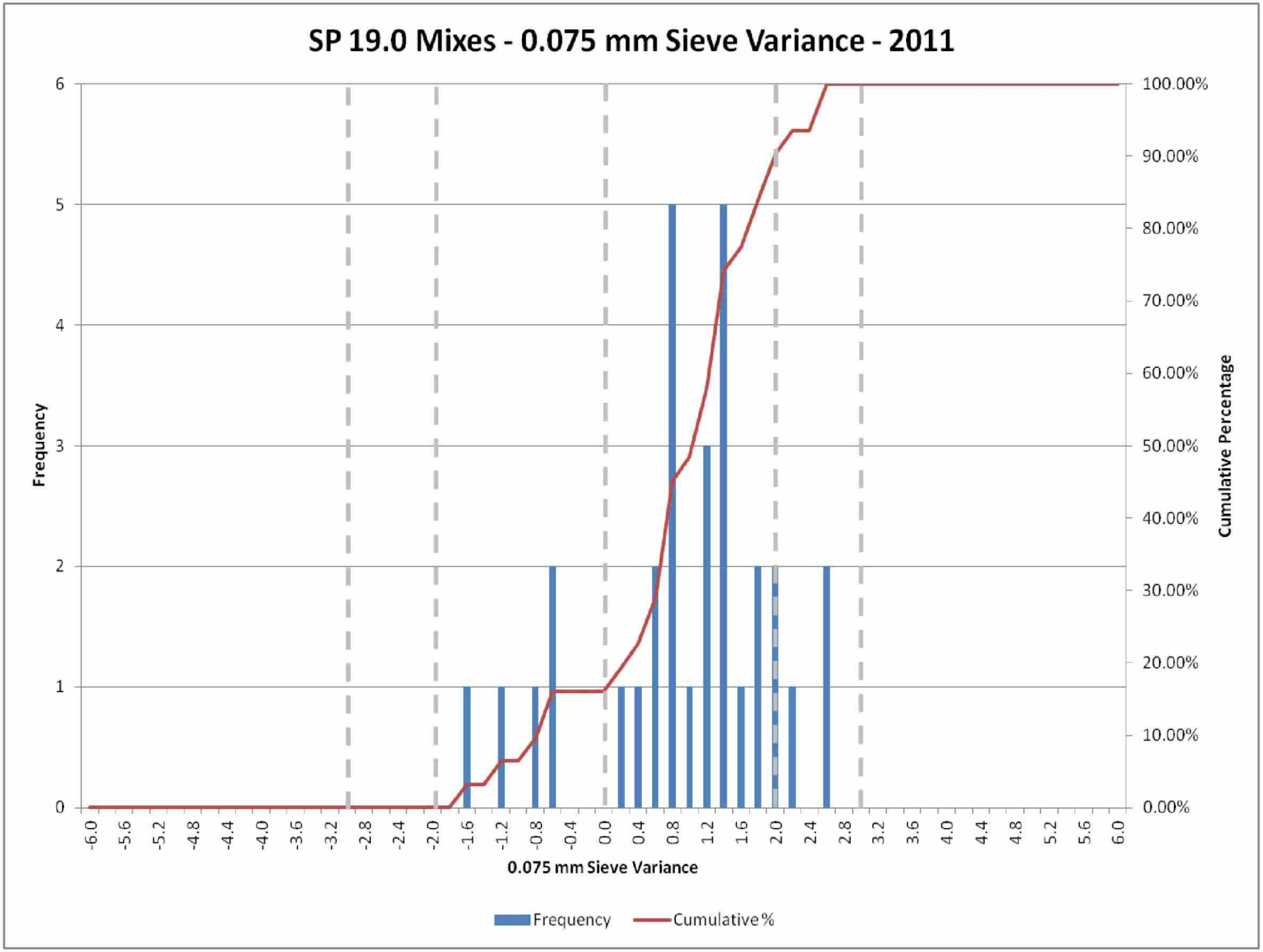

SP 12.5 Mixes - AC Content Variance - 2011
Frequency
Cumulative Percentage
AC Content Variance
10
9
8
7
6
5
4
3
2
1
0
100.00%
90.00%
80.00%
70.00%
60.00%
50.00%
40.00%
30.00%
20.00%
10.00%
0.00%
-0.80
-0.75
-0.70
-0.65
-0.60
-0.55
-0.50
-0.45
-0.40
-0.35
-0.30
-0.25
-0.20
-0.15
-0.10
-0.05
0.00
0.05
0.10
0.15
0.20
0.25
0.30
0.35
0.40
0.45
0.50
0.55
0.60
0.65
0.70
0.75
0.80
Frequency
Cumulative %

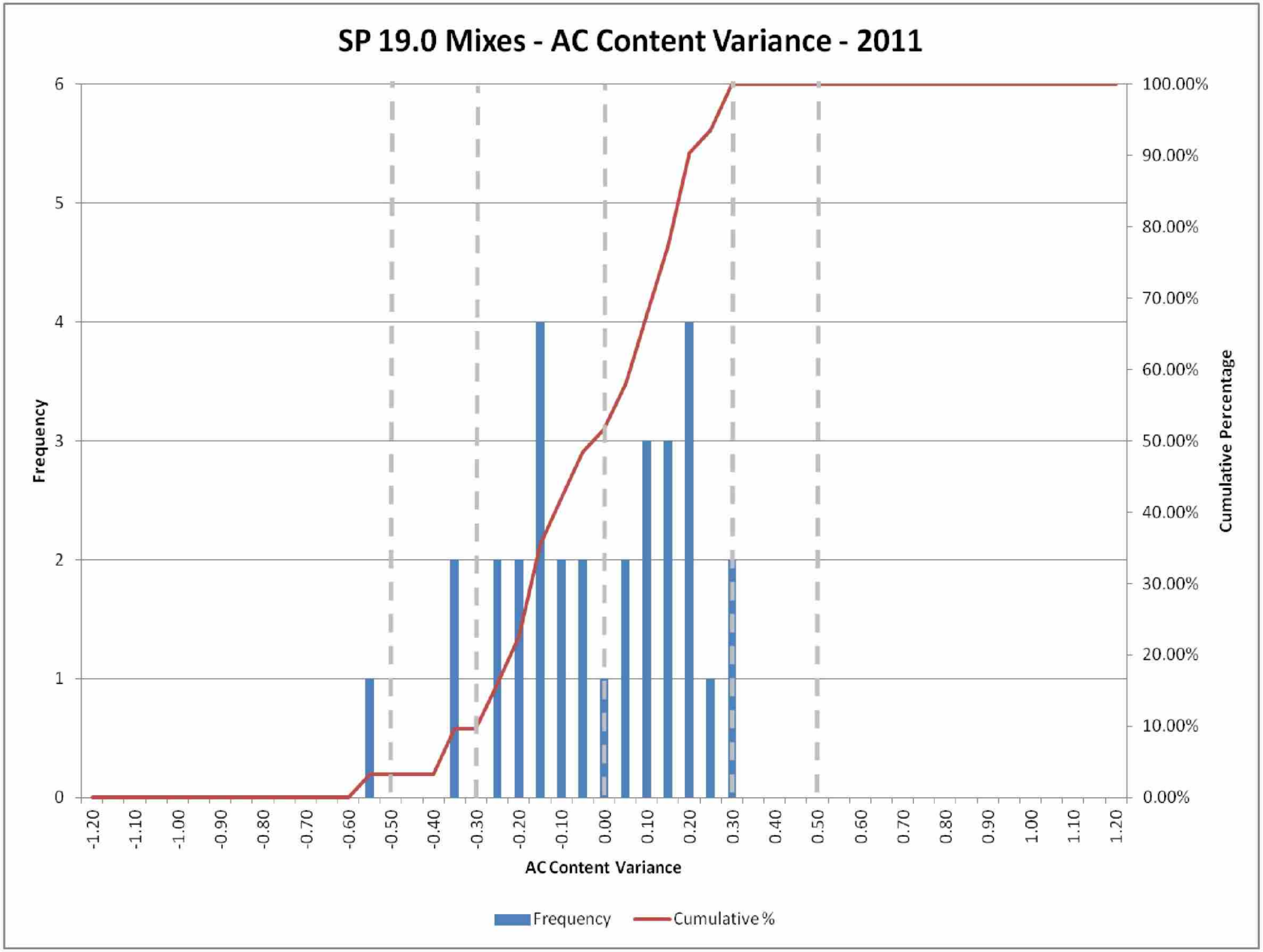

SP 19.0 Mixes - 4.75 mm Sieve Variance - 2011
Frequency
Cumulative Percentage
4.75 mm Sieve Variance
Frequency
Cumulative %
0
1
2
3
4
5
0.00%
10.00%
20.00%
30.00%
40.00%
50.00%
60.00%
70.00%
80.00%
90.00%
100.00%
-23.0 -22.0 -21.0 -20.0 -19.0 -18.0 -17.0 -16.0 -15.0 -14.0 -13.0 -12.0 -11.0 -10.0 -9.0 -8.0 -7.0 -6.0 -5.0 -4.0 -3.0 -2.0 -1.0 0.0 1.0 2.0 3.0 4.0 5.0 6.0 7.0 8.0 9.0 10.0 11.0 12.0 13.0 14.0 15.0 16.0 17.0 18.0 19.0 20.0 21.0 22.0 23.0



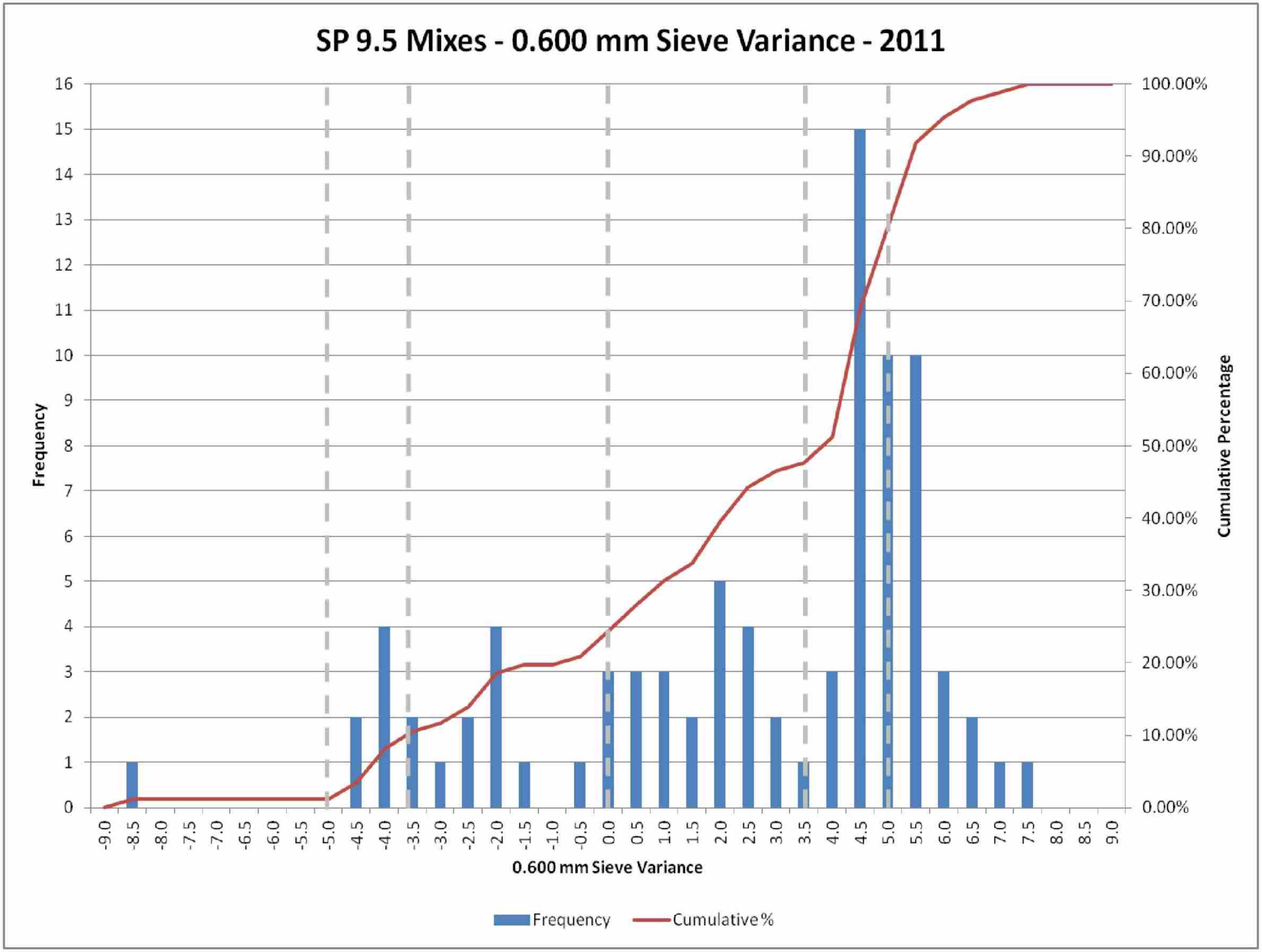
SP 19.0 Mixes - 0.600 mm Sieve Variance - 2011
Frequency
Cumulative Percentage
0.600 mm Sieve Variance
Frequency
Cumulative %

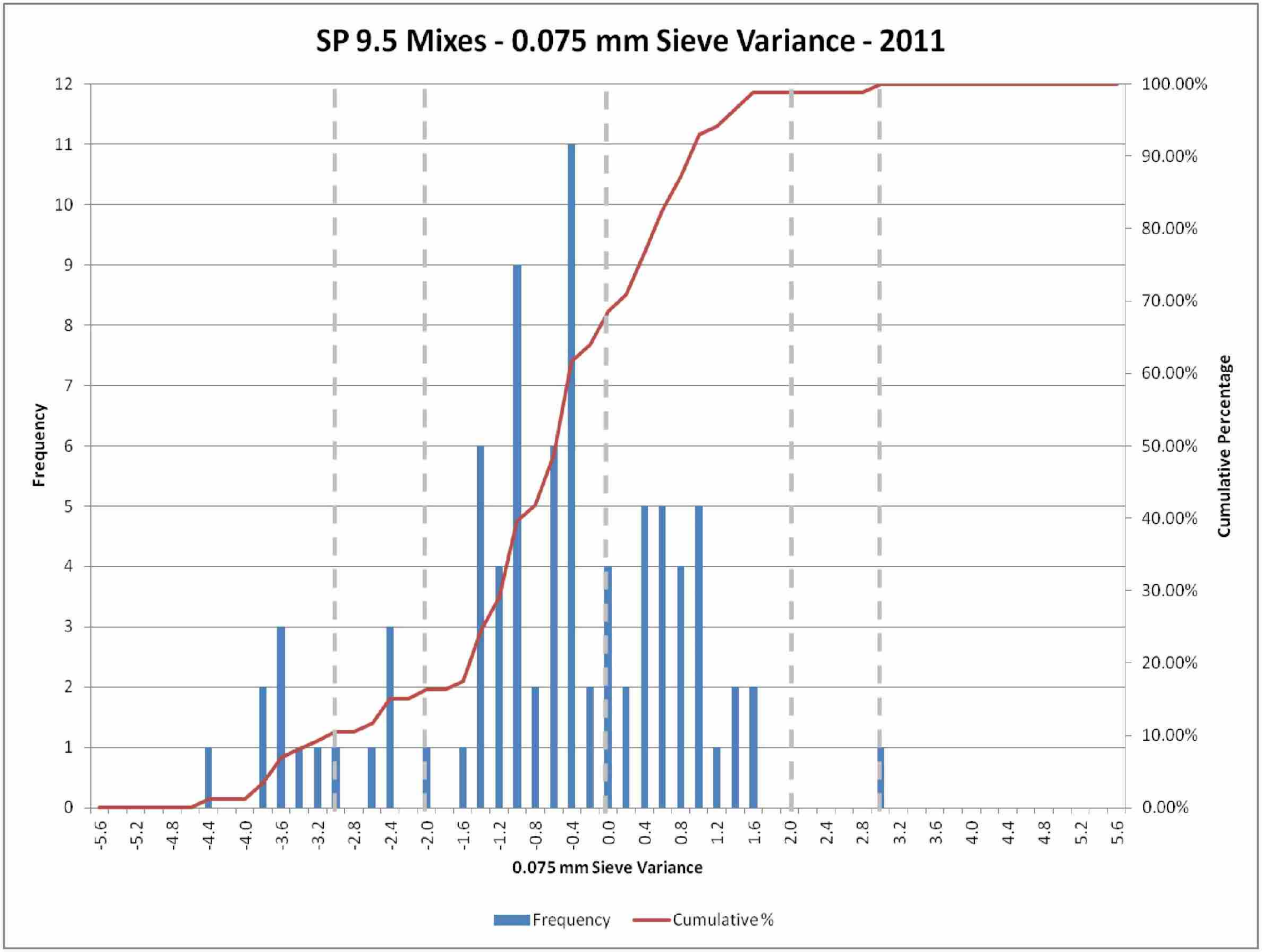
GOL0007504

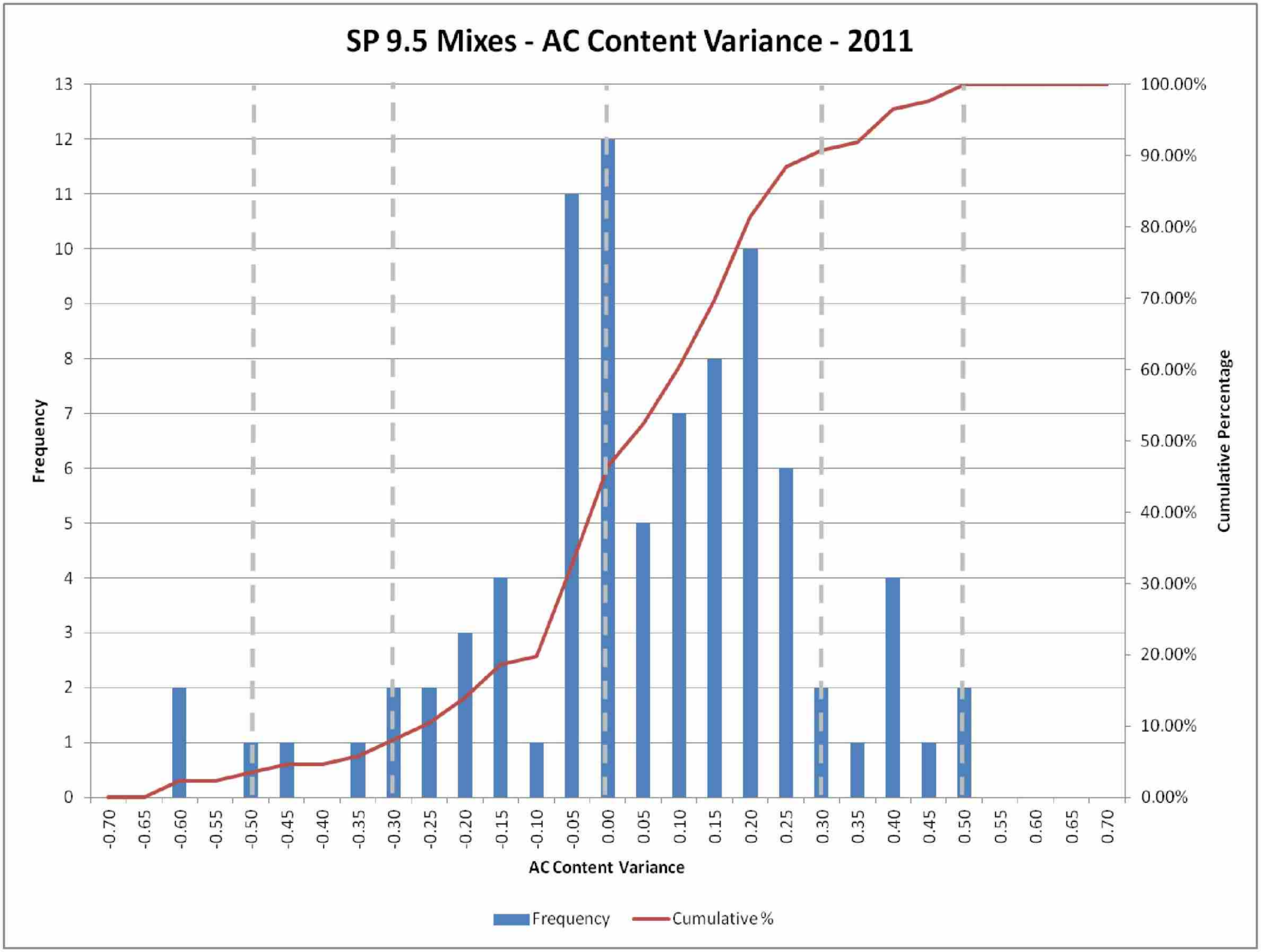

SP 19.0 Mixes - 0.075 mm Sieve Variance - 2011
Frequency
Cumulative Percentage
0.075 mm Sieve Variance
0
1
2
3
4
5
6
0.00%
10.00%
20.00%
30.00%
40.00%
50.00%
60.00%
70.00%
80.00%
90.00%
100.00%
-6.0
-5.6
-5.2
-4.8
-4.4
-4.0
-3.6
-3.2
-2.8
-2.4
-2.0
-1.6
-1.2
-0.8
-0.4
0.0
0.4
0.8
1.2
1.6
2.0
2.4
2.8
3.2
3.6
4.0
4.4
4.8
5.2
5.6
6.0
Frequency
Cumulative %



SP 19.0 Mixes - AC Content Variance - 2011
Frequency
Cumulative Percentage
AC Content Variance
0
1
2
3
4
5
6
0.00%
10.00%
20.00%
30.00%
40.00%
50.00%
60.00%
70.00%
80.00%
90.00%
100.00%
-1.20
-1.10
-1.00
-0.90
-0.80
-0.70
-0.60
-0.50
-0.40
-0.30
-0.20
-0.10
0.00
0.10
0.20
0.30
0.40
0.50
0.60
0.70
0.80
0.90
1.00
1.10
1.20
Frequency
Cumulative %


SP 9.5 Mixes - 0.600 mm Sieve Variance - 2011
Frequency
Cumulative Percentage
0.600 mm Sieve Variance
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16
0.00% 10.00% 20.00% 30.00% 40.00% 50.00% 60.00% 70.00% 80.00% 90.00% 100.00%
-9.0 -8.5 -8.0 -7.5 -7.0 -6.5 -6.0 -5.5 -5.0 -4.5 -4.0 -3.5 -3.0 -2.5 -2.0 -1.5 -1.0 -0.5 0.0 0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0 4.5 5.0 5.5 6.0 6.5 7.0 7.5 8.0 8.5 9.0
Frequency
Cumulative %


SP 9.5 Mixes - 0.075 mm Sieve Variance - 2011
Frequency
Cumulative Percentage
0.075 mm Sieve Variance
Frequency
Cumulative %
12
11
10
9
8
7
6
5
4
3
2
1
0
100.00%
90.00%
80.00%
70.00%
60.00%
50.00%
40.00%
30.00%
20.00%
10.00%
0.00%
-5.6
-5.2
-4.8
-4.4
-4.0
-3.6
-3.2
-2.8
-2.4
-2.0
-1.6
-1.2
-0.8
-0.4
0.0
0.4
0.8
1.2
1.6
2.0
2.4
2.8
3.2
3.6
4.0
4.4
4.8
5.2
5.6



SP 9.5 Mixes - AC Content Variance - 2011
Frequency
Cumulative Percentage
AC Content Variance
13
12
11
10
9
8
7
6
5
4
3
2
1
0
100.00%
90.00%
80.00%
70.00%
60.00%
50.00%
40.00%
30.00%
20.00%
10.00%
0.00%
-0.70
-0.65
-0.60
-0.55
-0.50
-0.45
-0.40
-0.35
-0.30
-0.25
-0.20
-0.15
-0.10
-0.05
0.00
0.05
0.10
0.15
0.20
0.25
0.30
0.35
0.40
0.45
0.50
0.55
0.60
0.65
0.70
Frequency
Cumulative %

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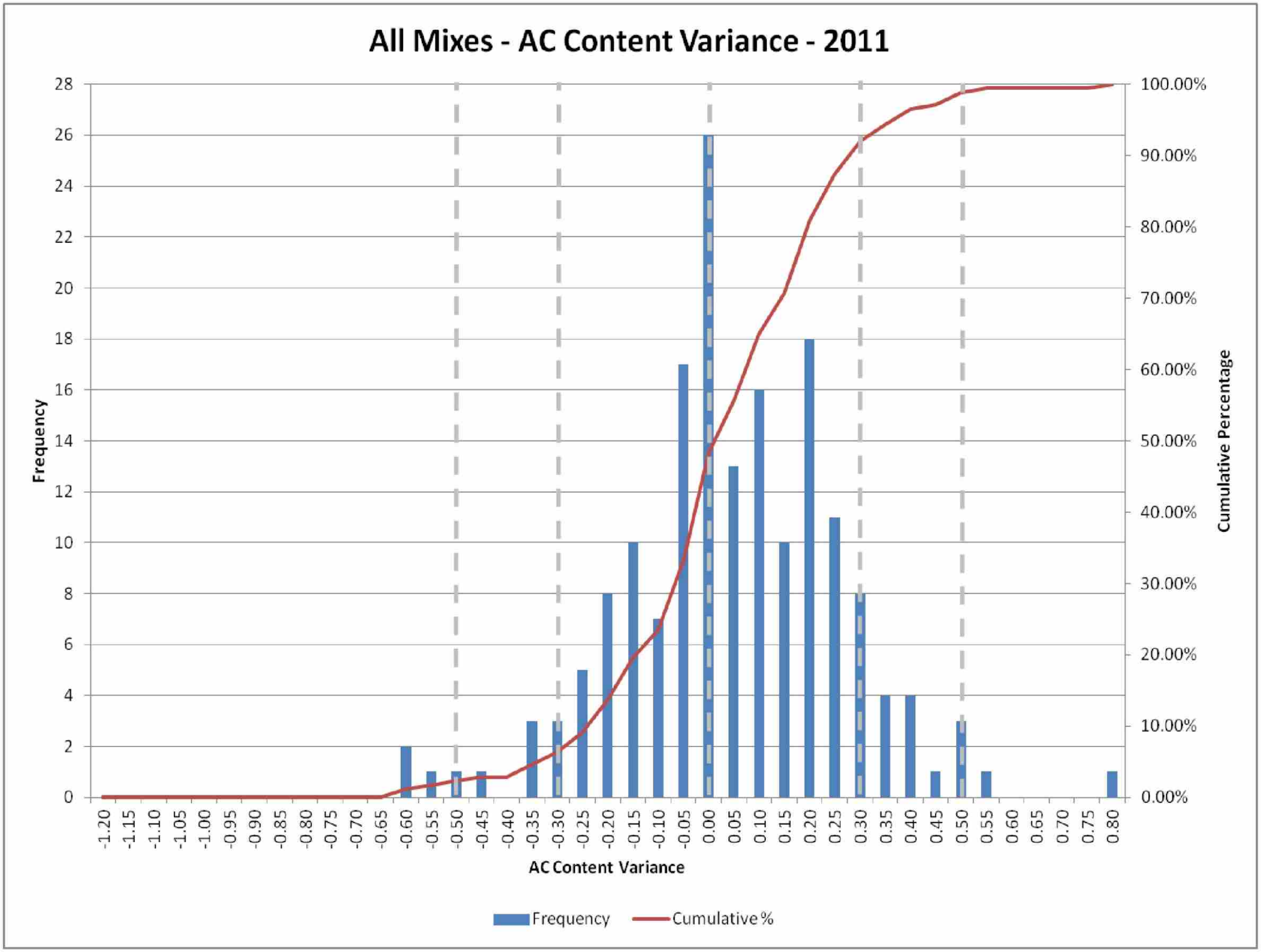
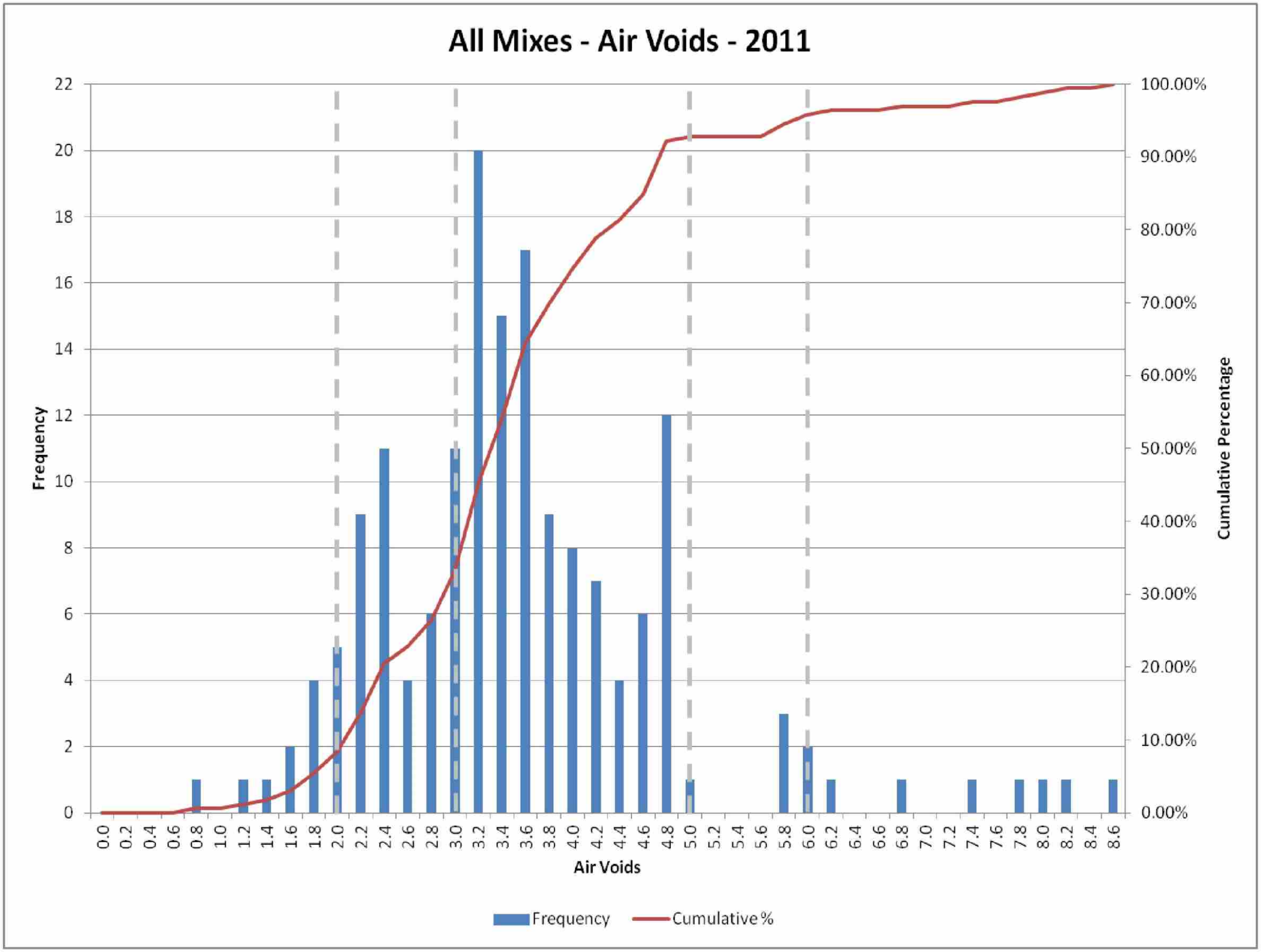
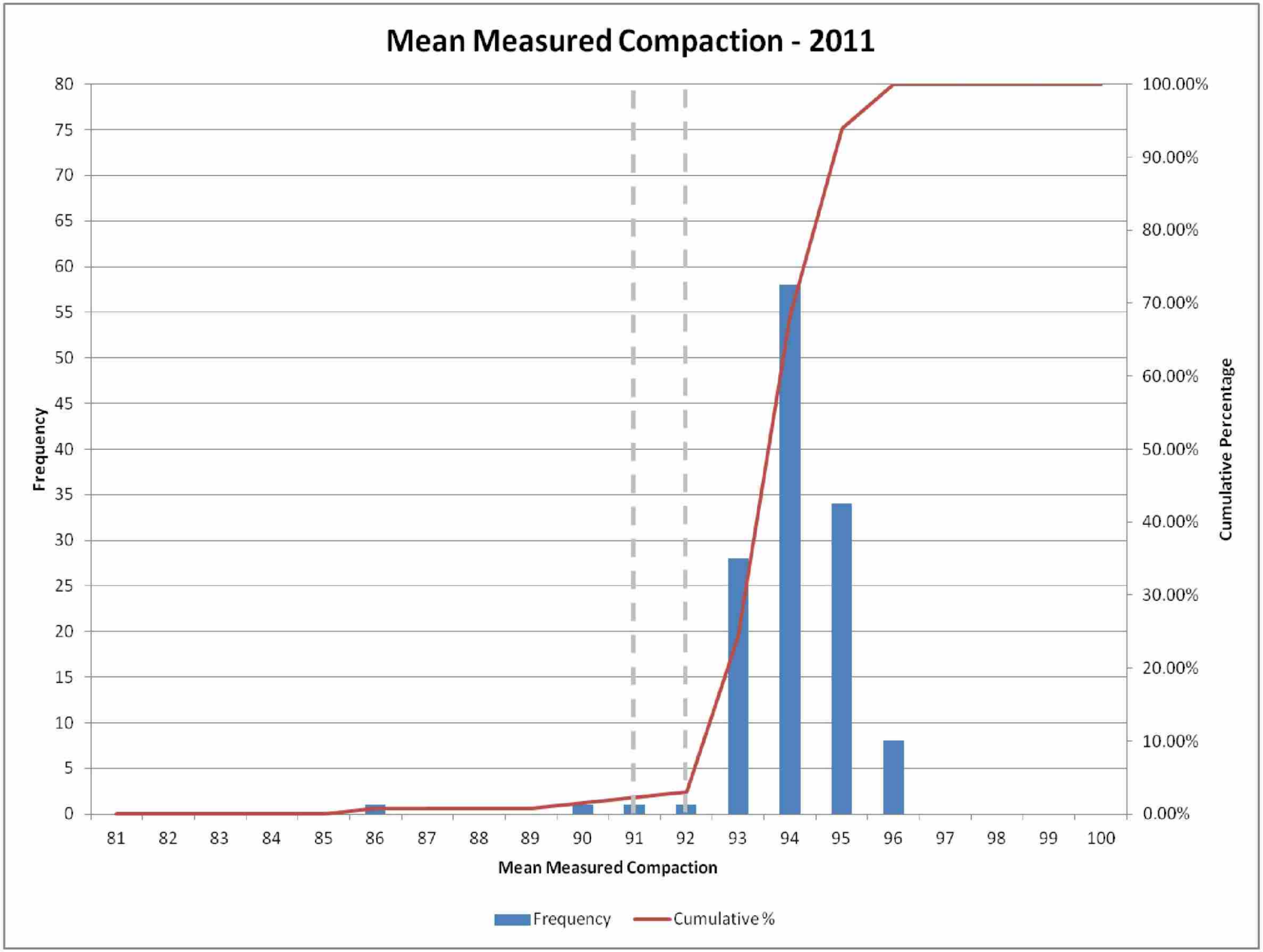

All Mixes - AC Content Variance - 2011
Frequency
Cumulative Percentage
AC Content Variance
Frequency
Cumulative %

GOL0007504



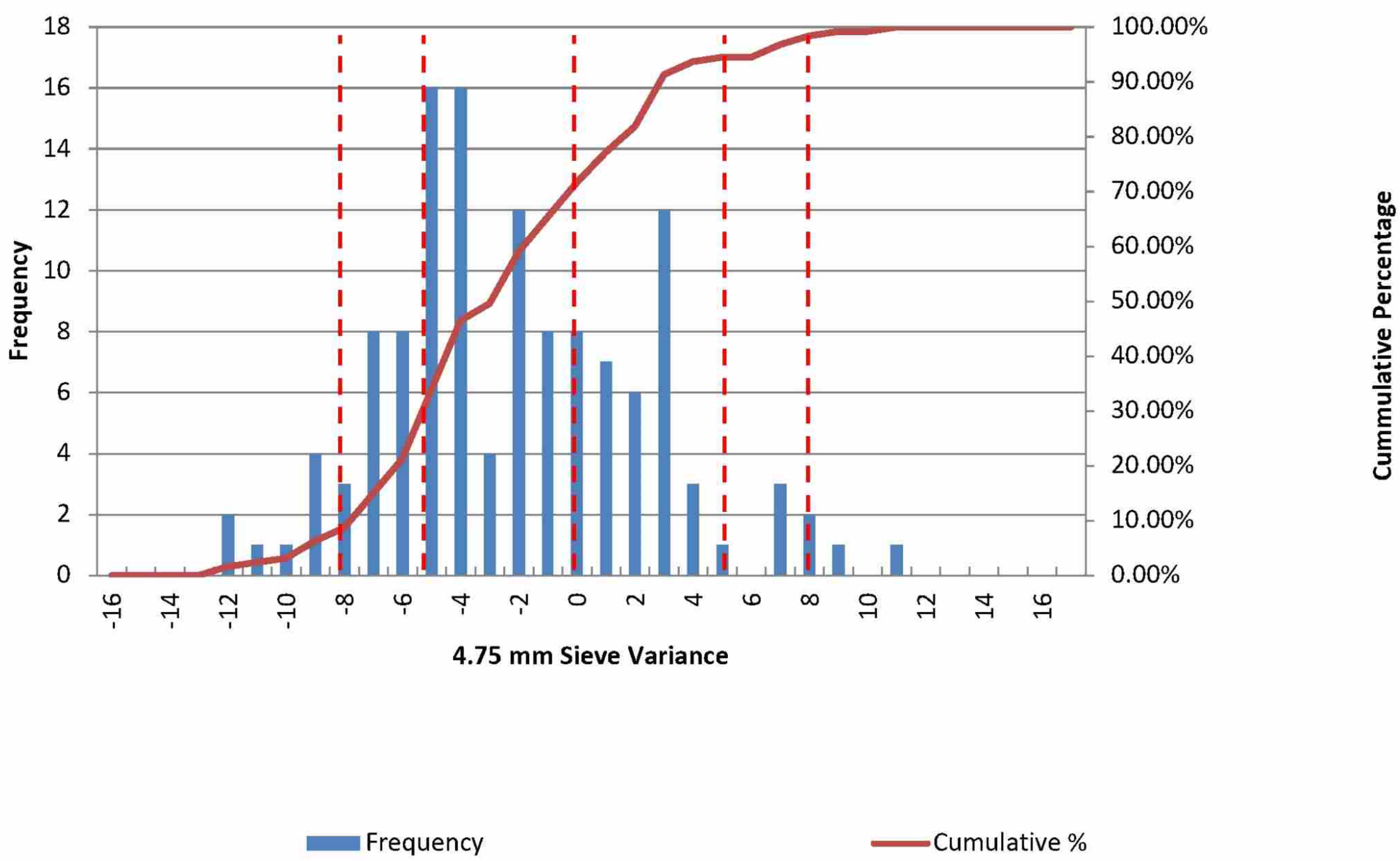
All Mixes - Air Voids - 2011
Frequency
Cumulative Percentage
Air Voids
22
20
18
16
14
12
10
8
6
4
2
0
100.00%
90.00%
80.00%
70.00%
60.00%
50.00%
40.00%
30.00%
20.00%
10.00%
0.00%
0.0 0.2 0.4 0.6 0.8 1.0 1.2 1.4 1.6 1.8 2.0 2.2 2.4 2.6 2.8 3.0 3.2 3.4 3.6 3.8 4.0 4.2 4.4 4.6 4.8 5.0 5.2 5.4 5.6 5.8 6.0 6.2 6.4 6.6 6.8 7.0 7.2 7.4 7.6 7.8 8.0 8.2 8.4 8.6
Frequency
Cumulative %


Mean Measured Compaction - 2011
Frequency
Cumulative Percentage
Mean Measured Compaction
80
75
70
65
60
55
50
45
40
35
30
25
20
15
10
5
0
100.00%
90.00%
80.00%
70.00%
60.00%
50.00%
40.00%
30.00%
20.00%
10.00%
0.00%
81 82 83 84 85 86 87 88 89 90 91 92 93 94 95 96 97 98 99 100
Frequency
Cumulative %

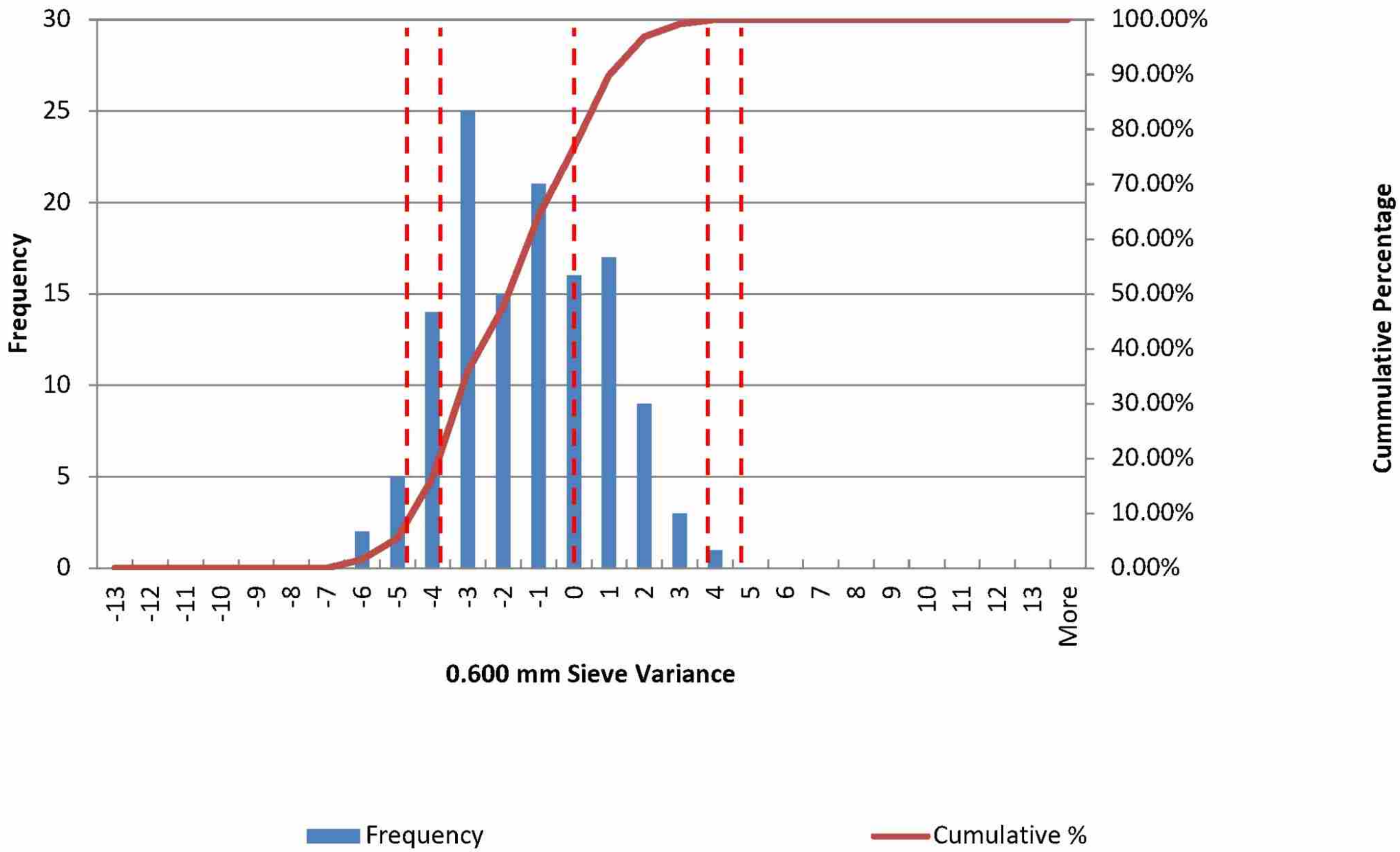
GOL0007504

2012

SP 12.5 Mixes - 4.75 mm Sieve Variance 2012



SP 12.5 Mixes - 0.600 mm Sieve Variance 2012



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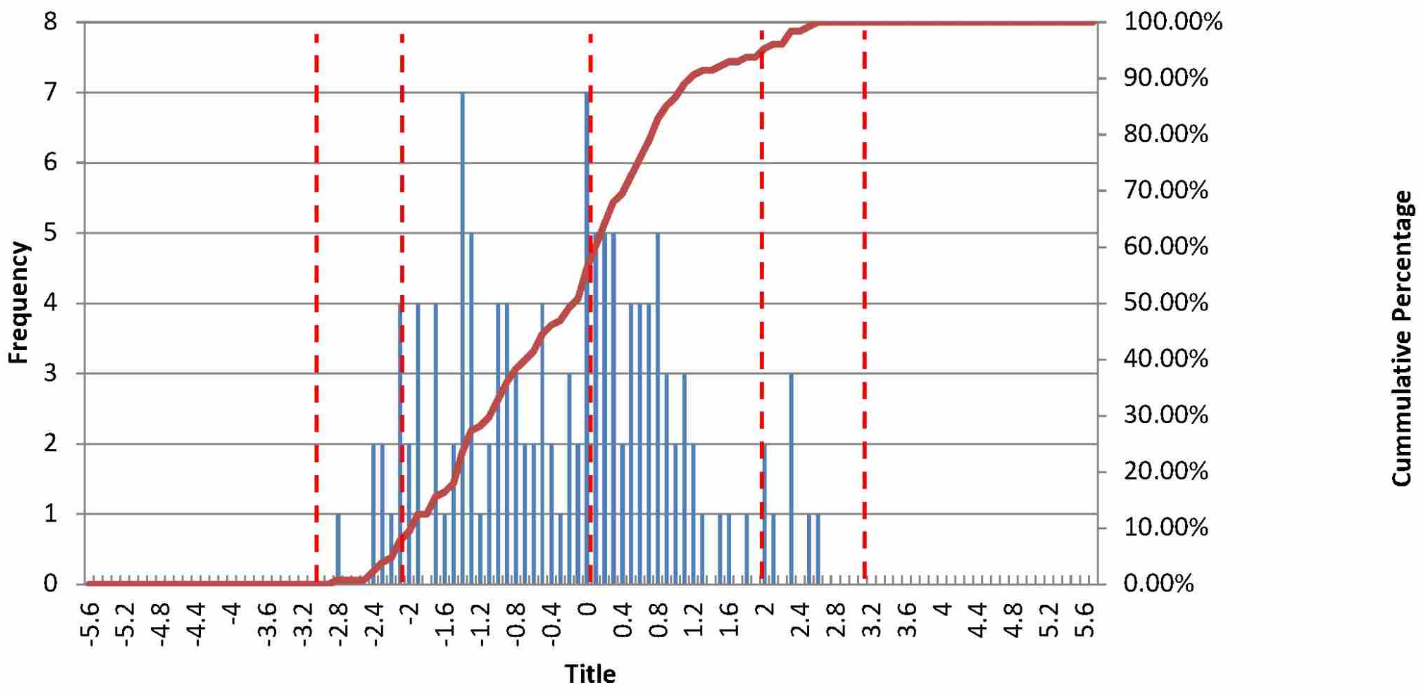
SP 12.5 Mixes - 0.075 mm Sieve Variance 2012



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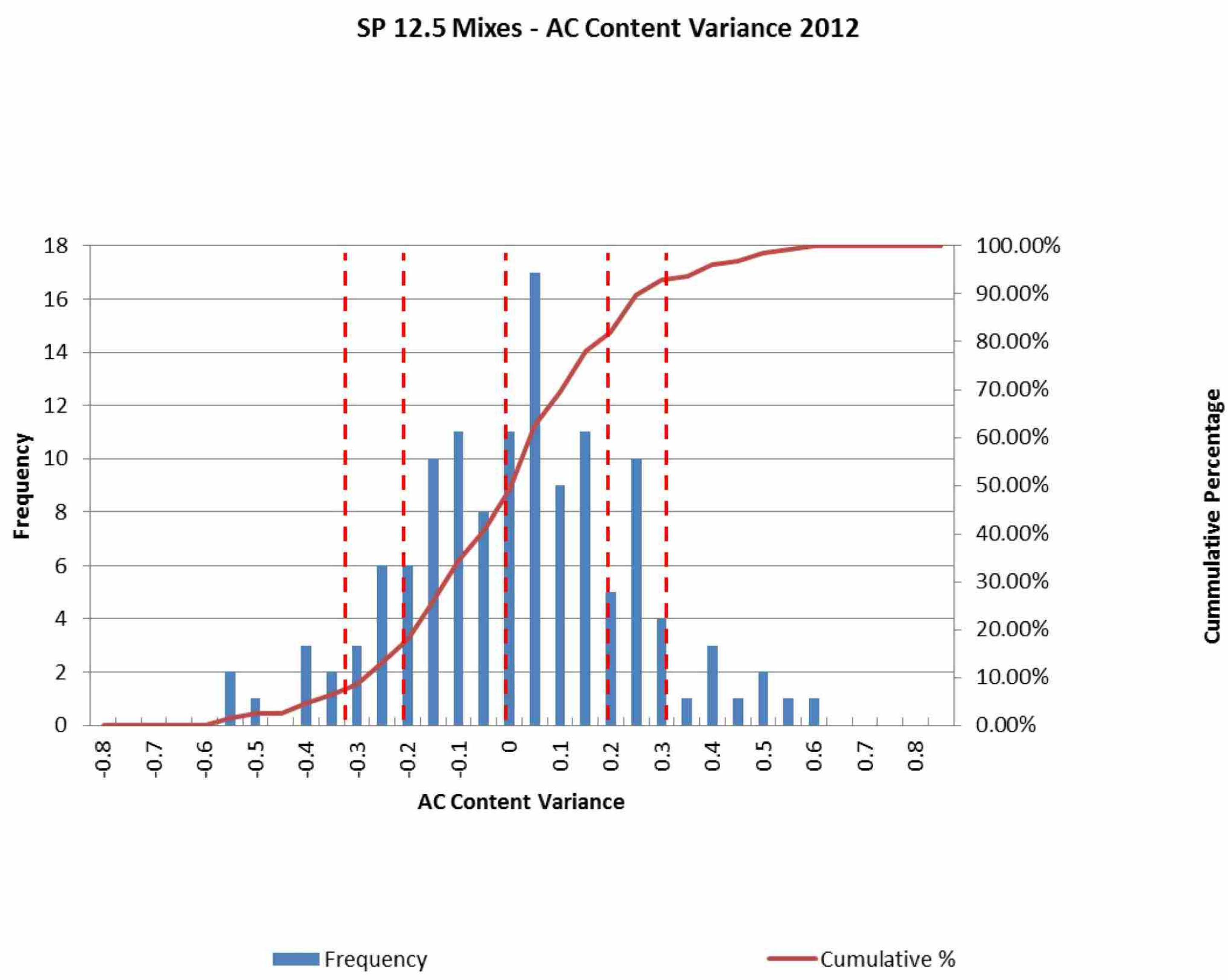
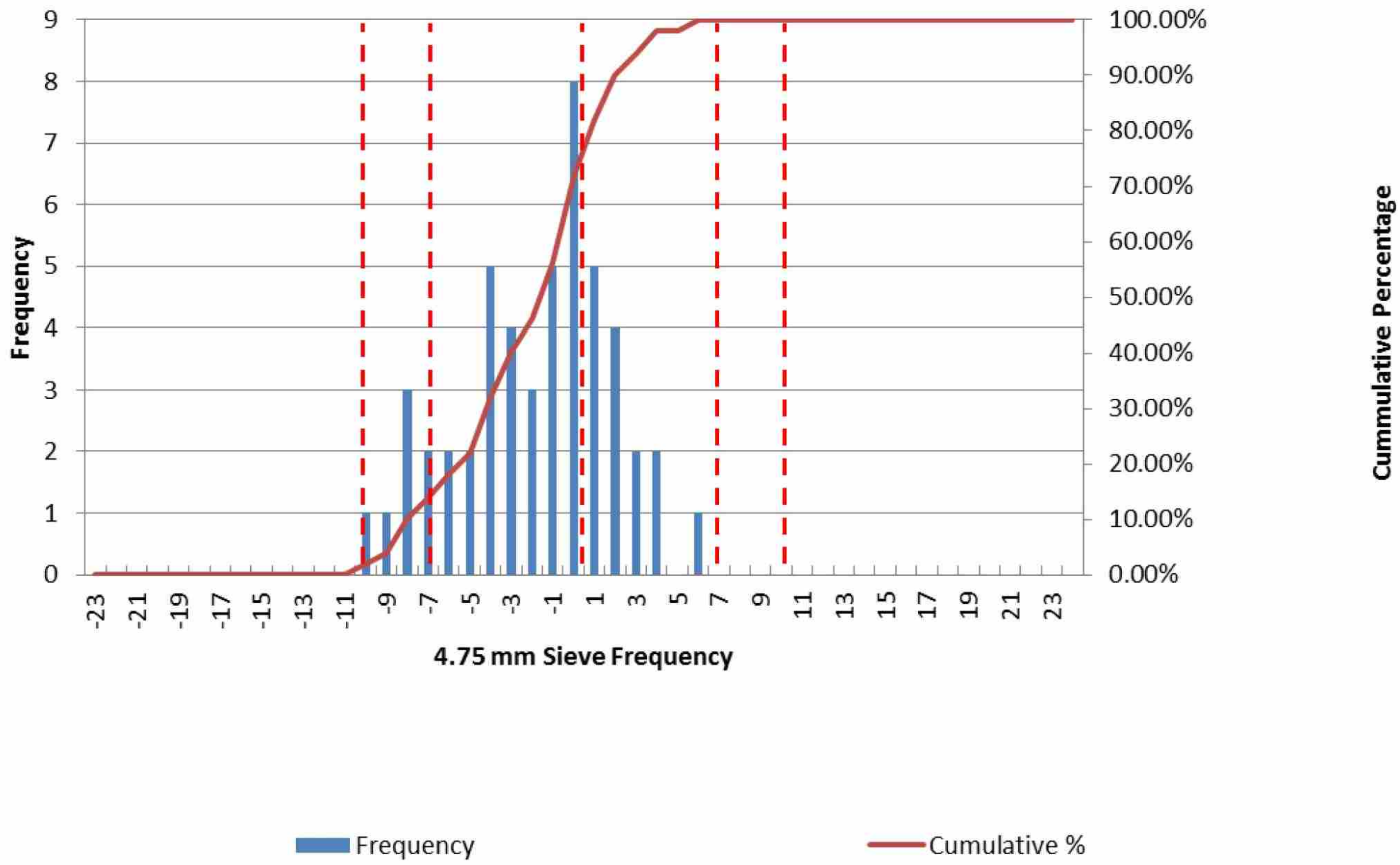
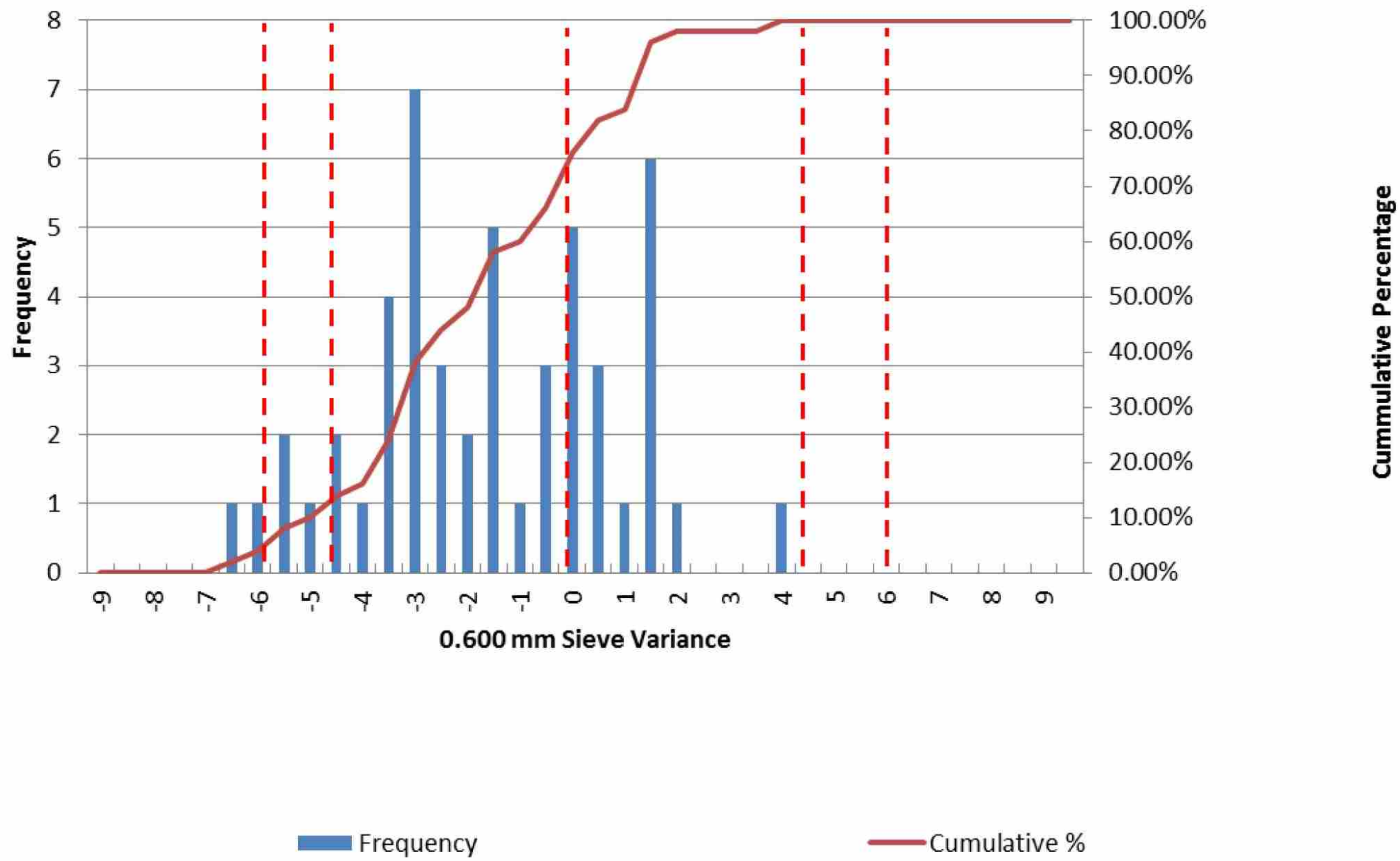

SP 12.5 Mixes - AC Content Variance 2012
Frequency
Cummulative Percentage
AC Content Variance
18
16
14
12
10
8
6
4
2
0
100.00%
90.00%
80.00%
70.00%
60.00%
50.00%
40.00%
30.00%
20.00%
10.00%
0.00%
-0.8
-0.7
-0.6
-0.5
-0.4
-0.3
-0.2
-0.1
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
Frequency
Cumulative %

GOL0007504

SP 19.0 Mixes - 4.75 mm Sieve Variance 2012

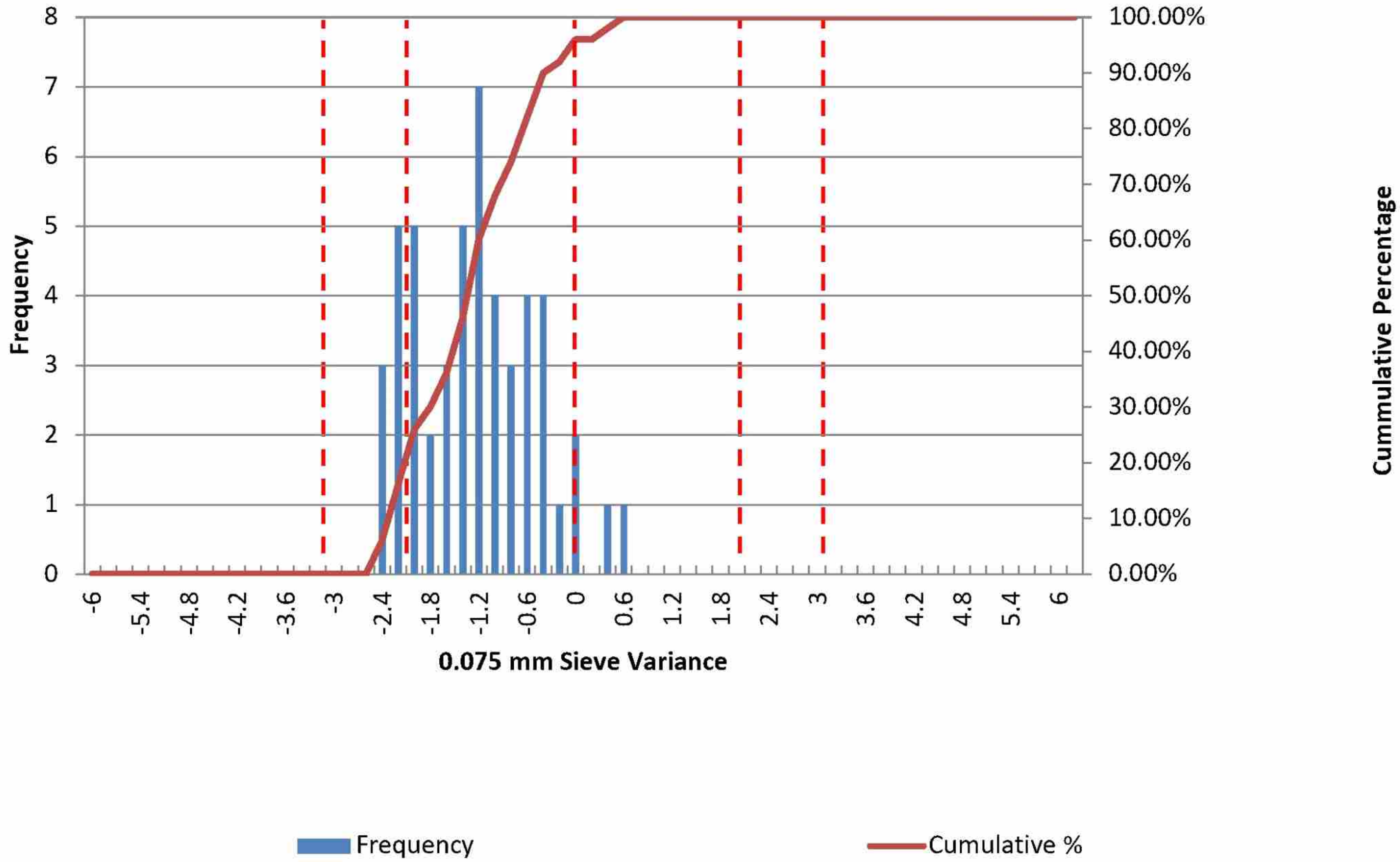


SP 19.0 Mixes - 0.600 mm Sieve Variance 2012

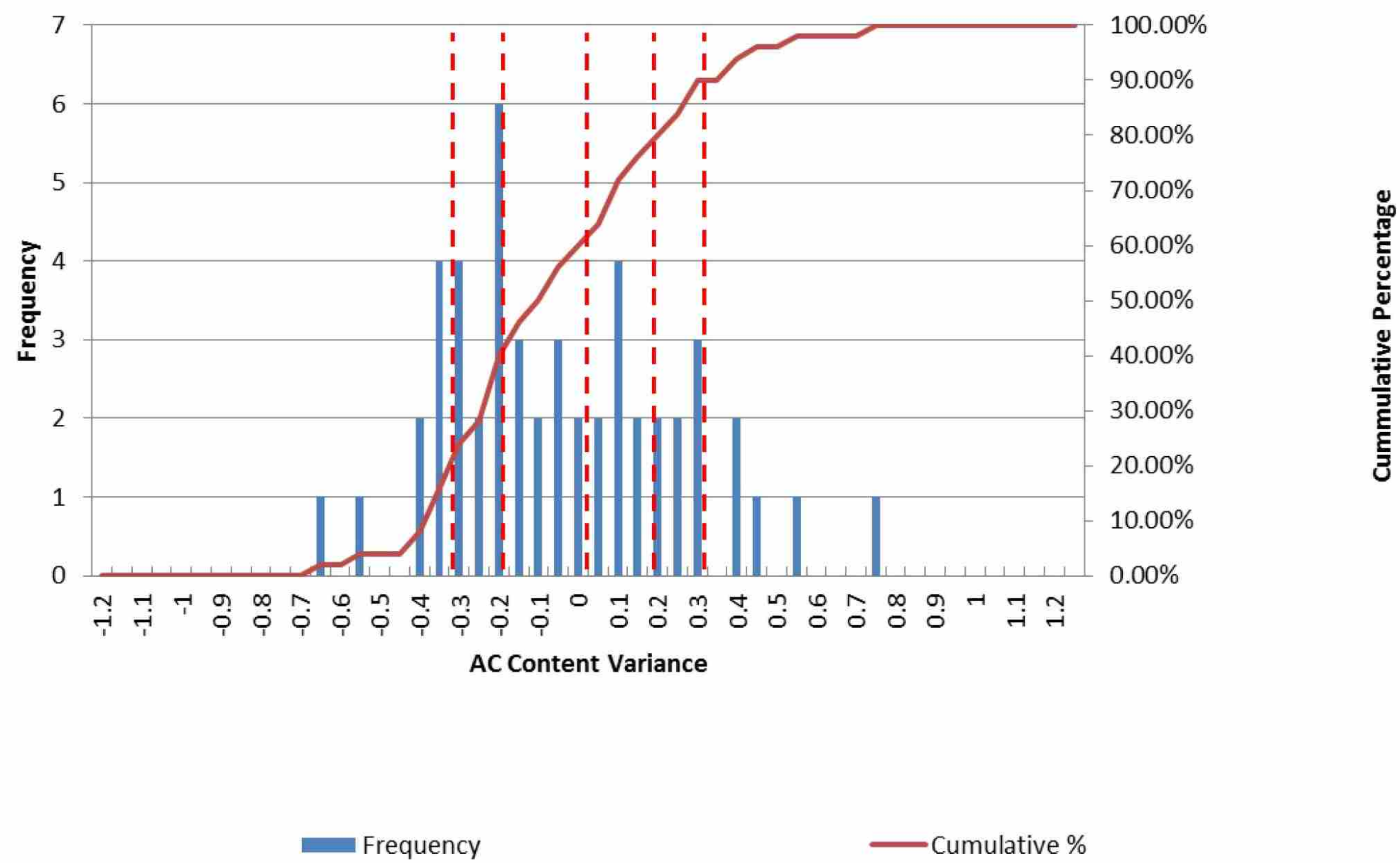


GOL0007504

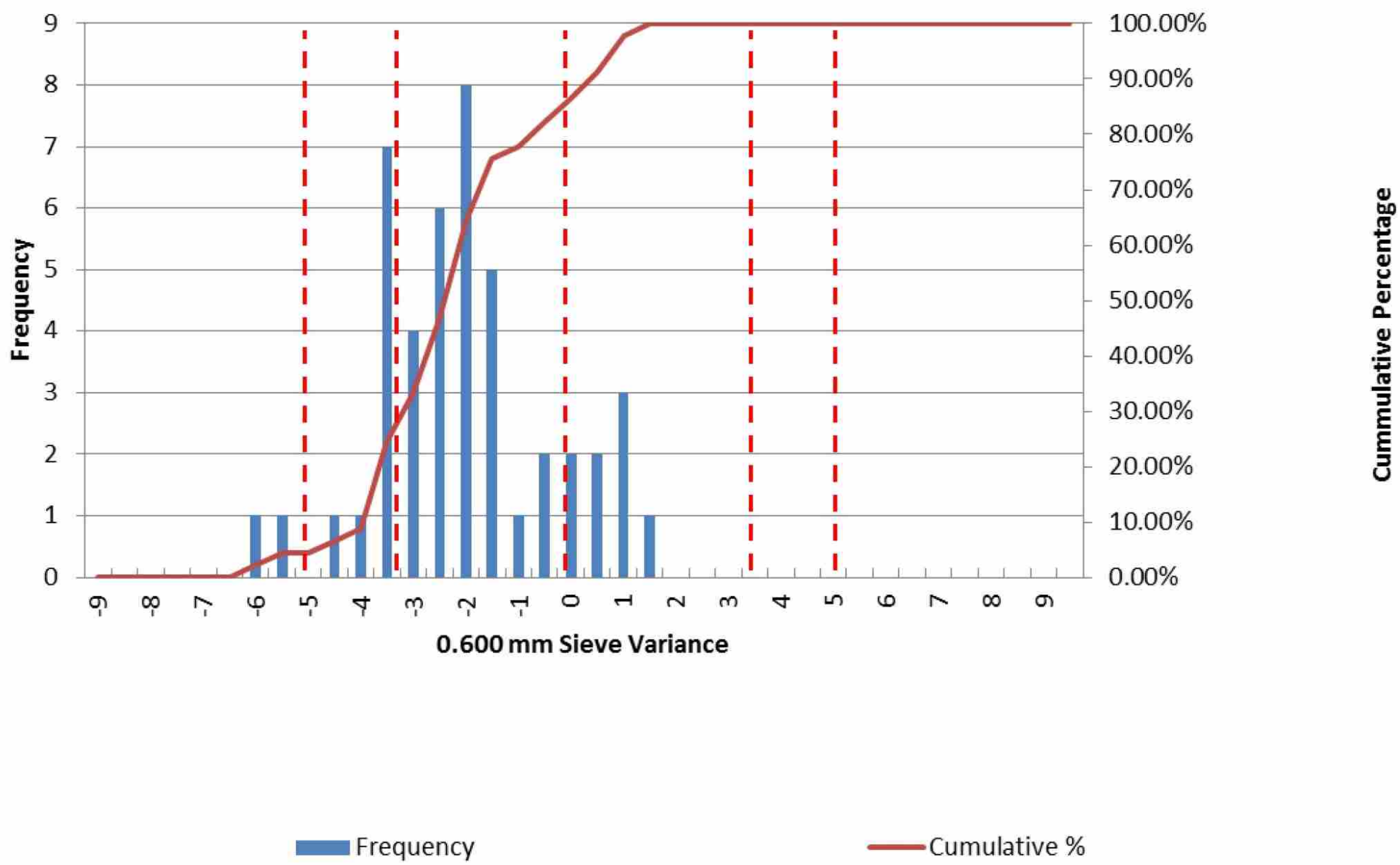
SP 19.0 mm Mixes - 0.075 mm Sieve Variance 2012



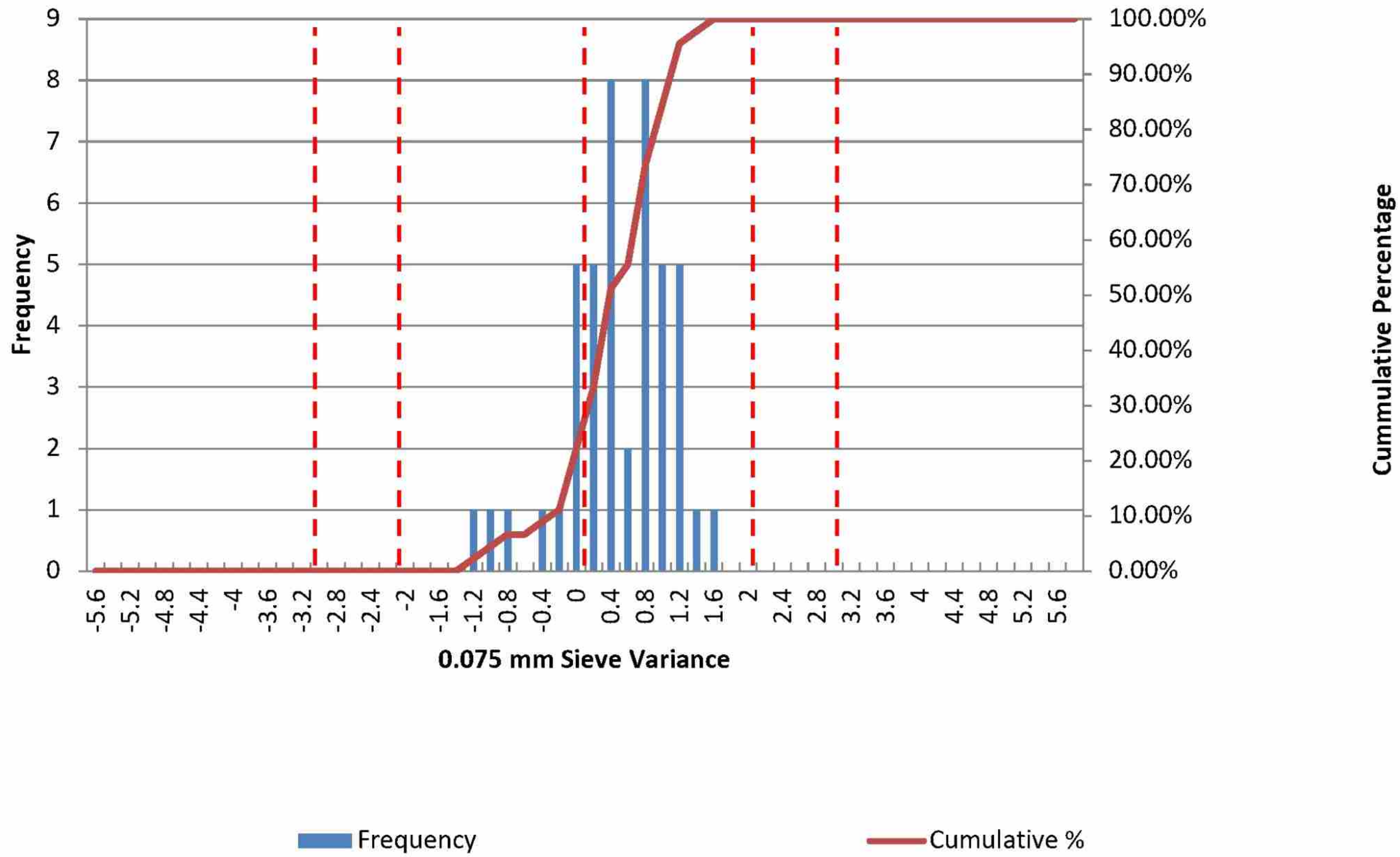
SP 19.0 Mixes - AC Content Variances 2012



SP 9.5 Mixes - 0.600 mm Sieve Variance 2012

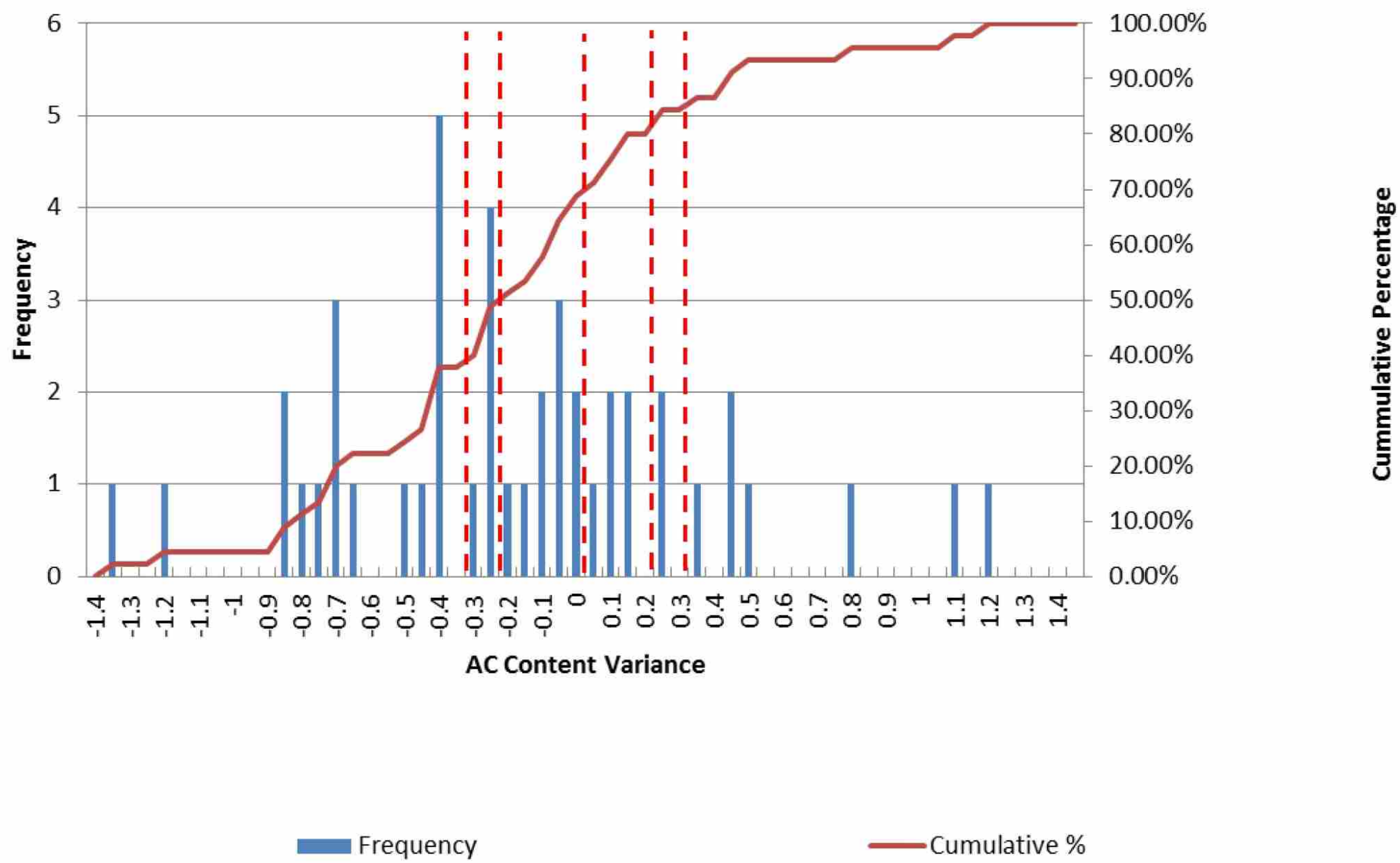


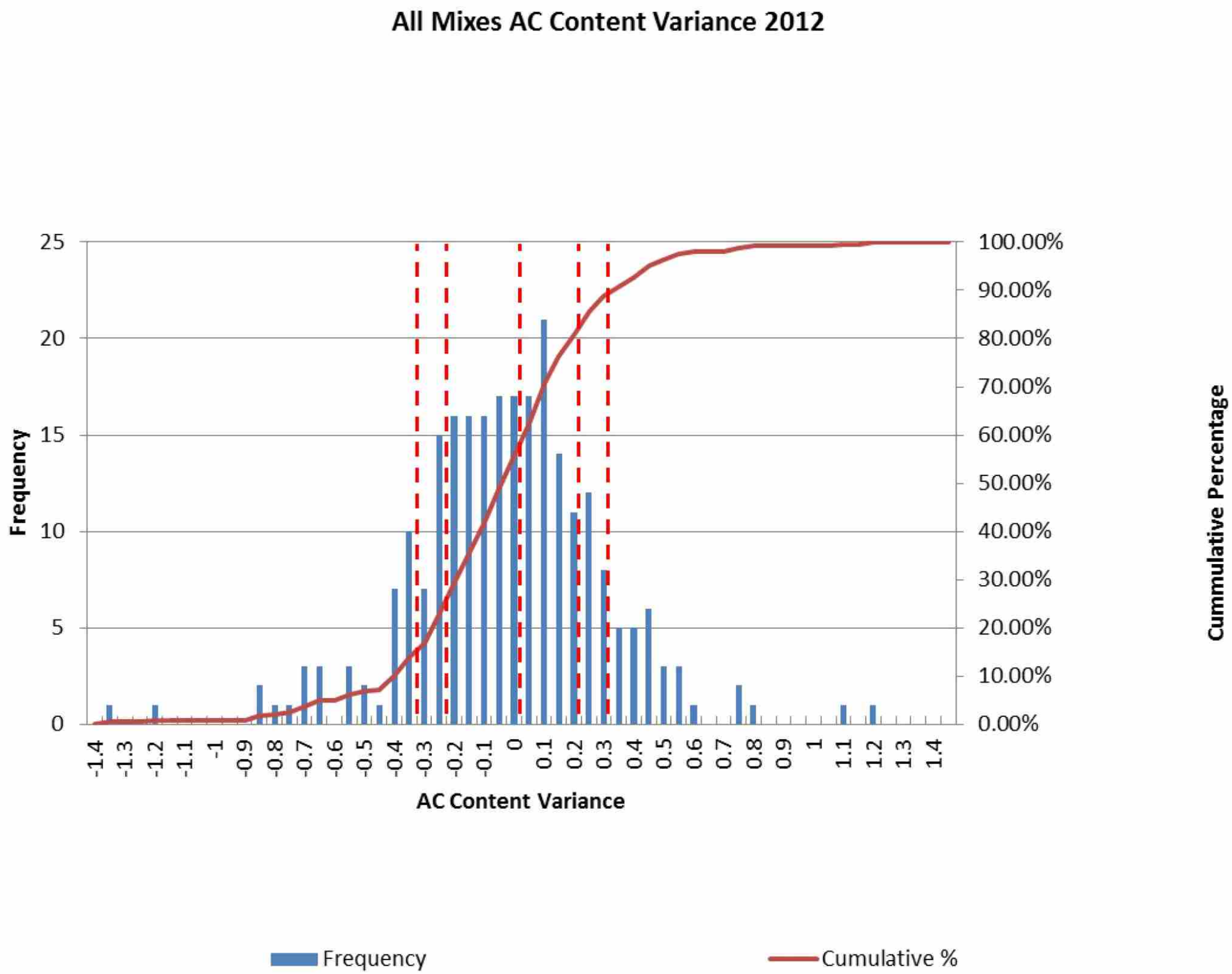
SP 9.5 mm Mixes - 0.075 mm Sieve Variance 2012



GOL0007504

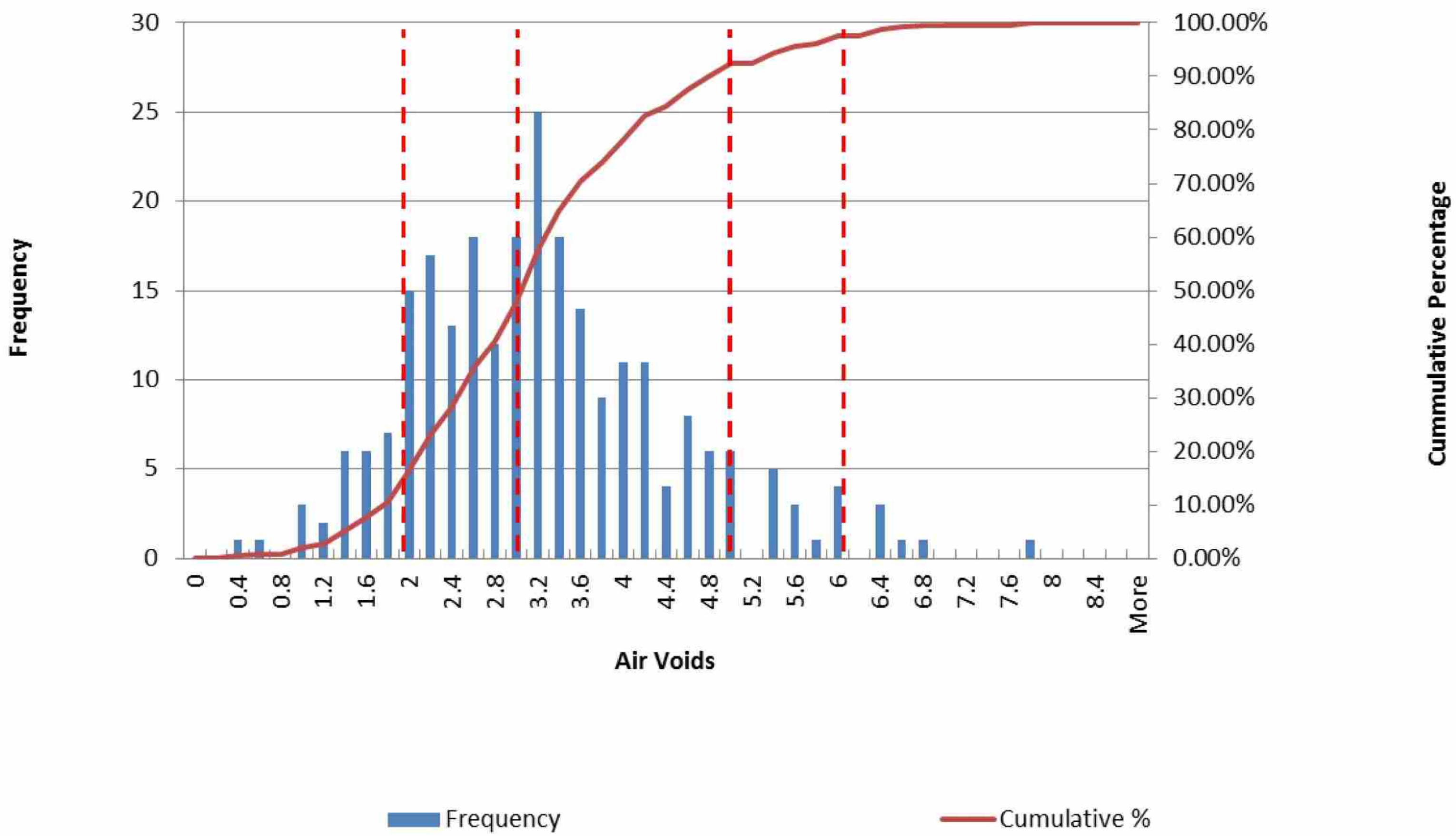
SP 9.5 Mixes - AC Content Variance 2012

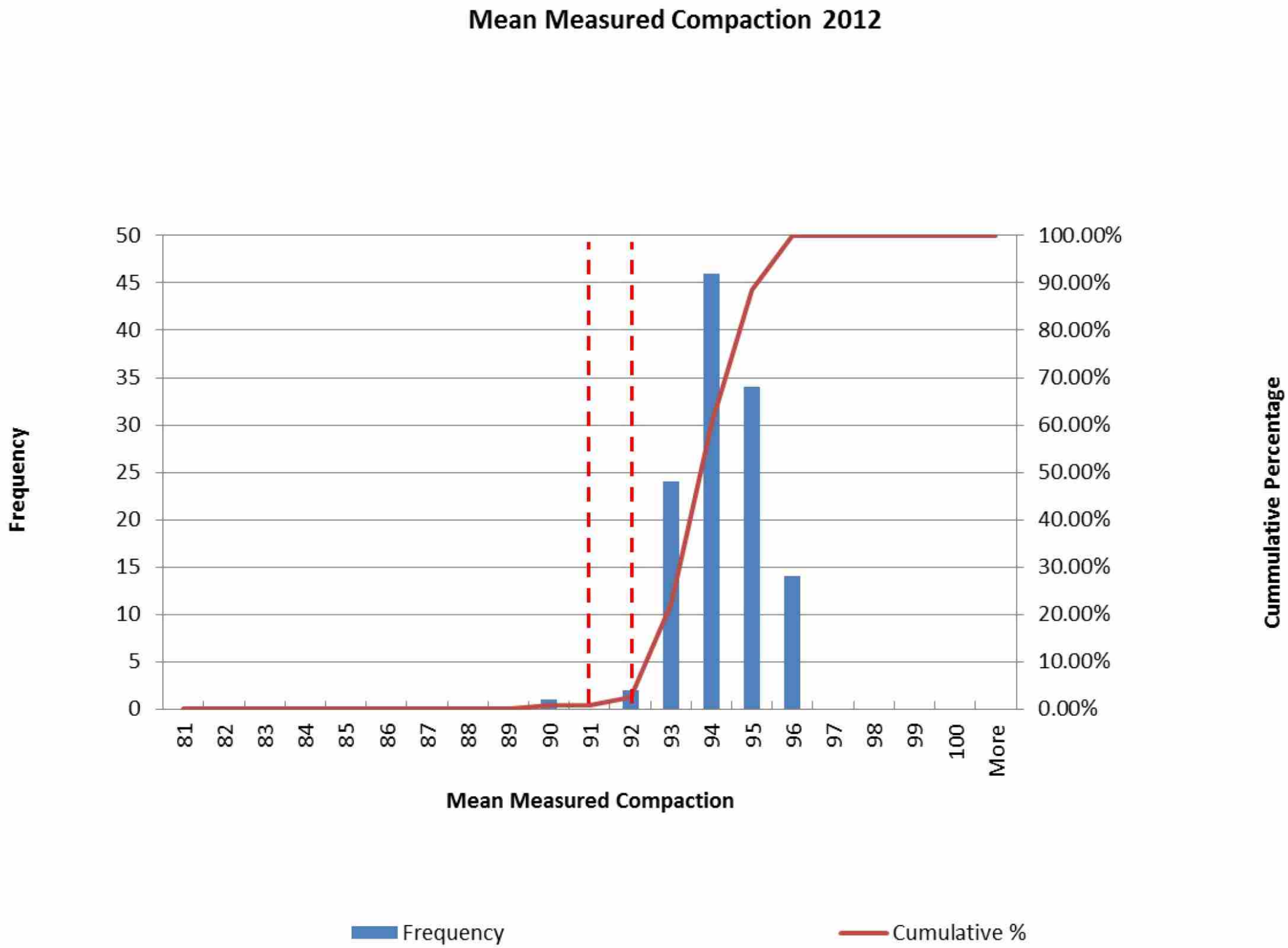



All Mixes AC Content Variance 2012
Frequency
25
20
15
10
5
0
100.00%
90.00%
80.00%
70.00%
60.00%
50.00%
40.00%
30.00%
20.00%
10.00%
0.00%
Cummulative Percentage
-1.4
-1.3
-1.2
-1.1
-1
-0.9
-0.8
-0.7
-0.6
-0.5
-0.4
-0.3
-0.2
-0.1
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
1.1
1.2
1.3
1.4
AC Content Variance
Frequency
Cumulative %

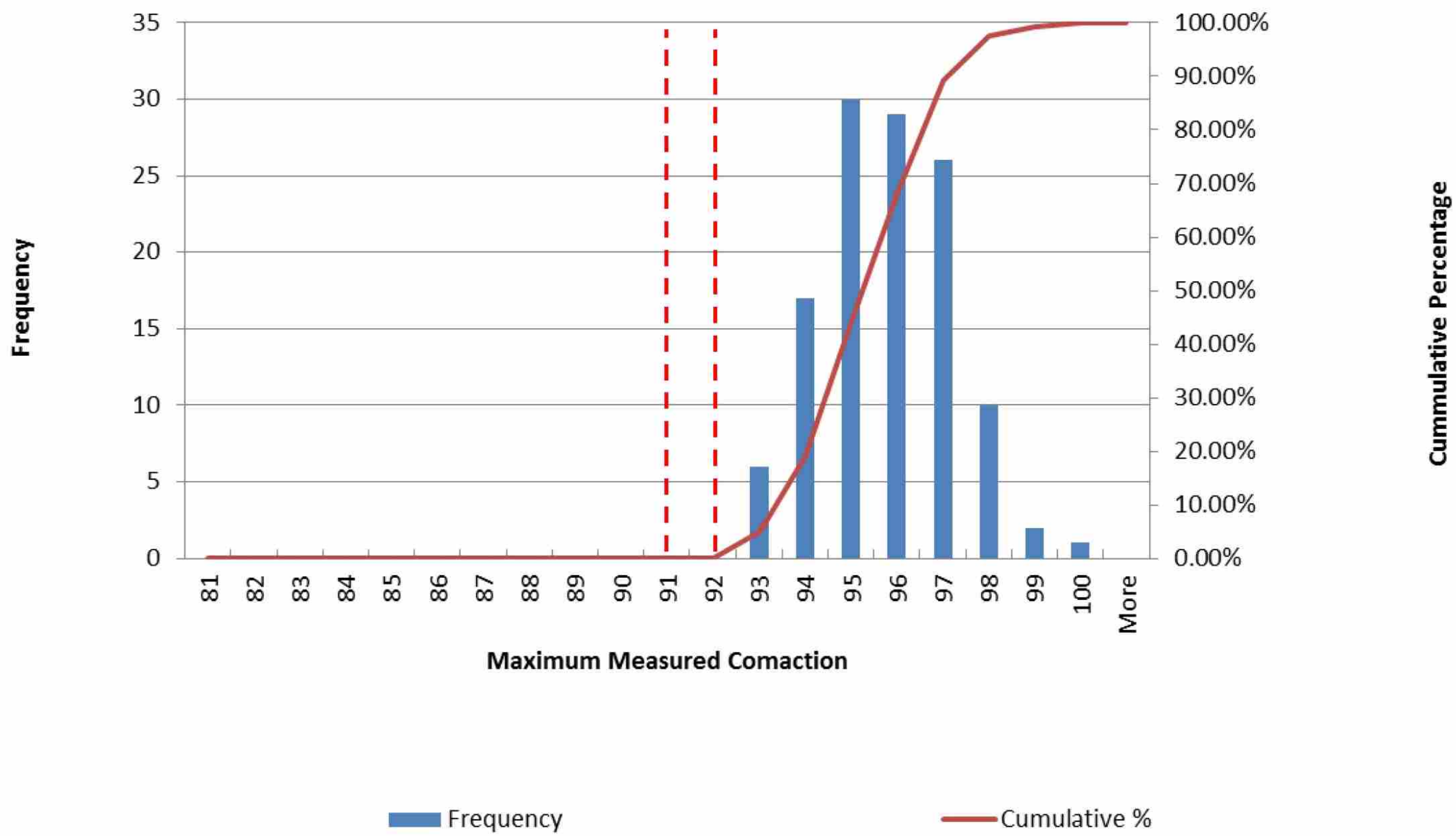
GOL0007504

All Mixes Air Voids 2012

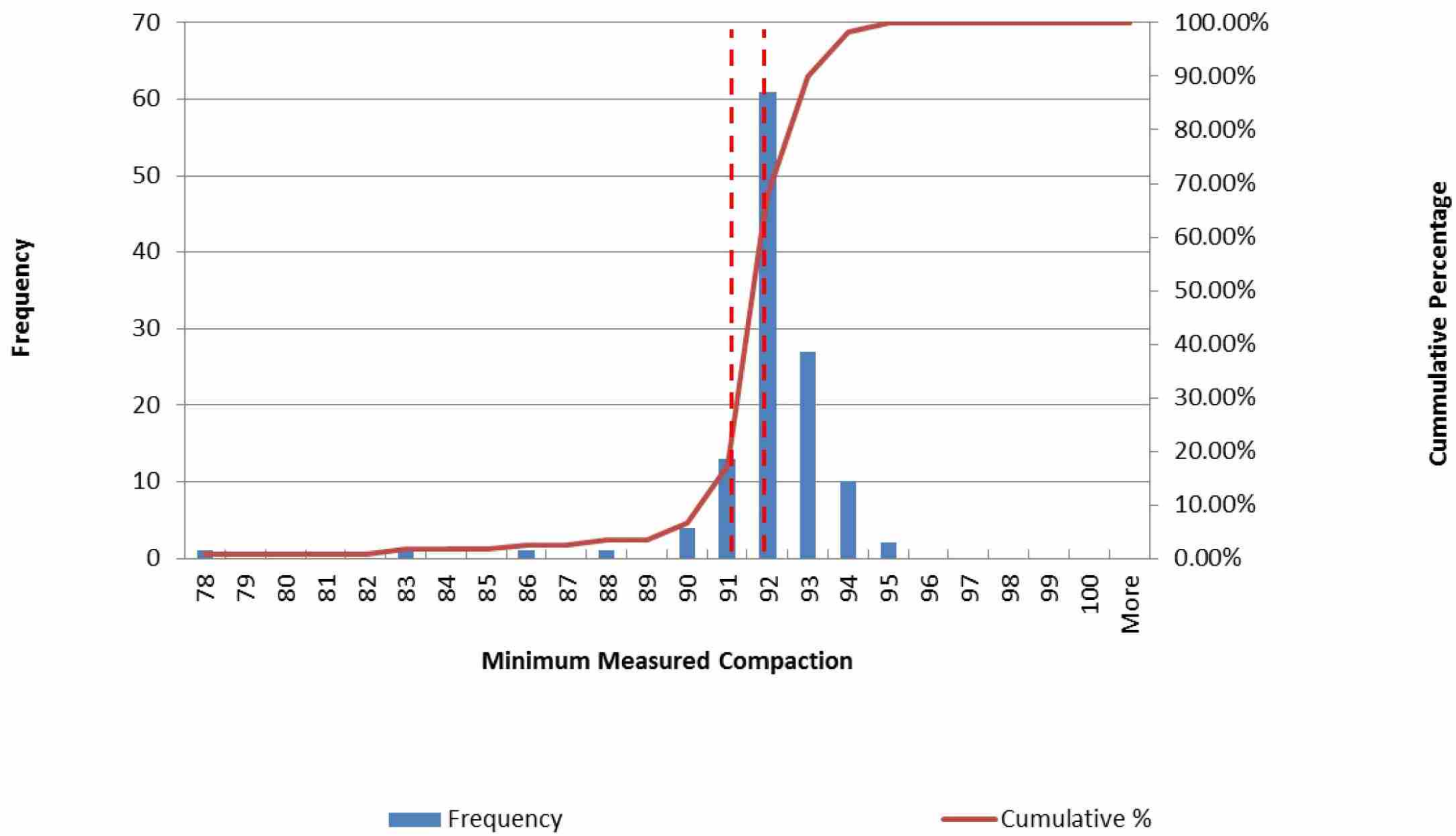



Mean Measured Compaction 2012
Frequency
50
45
40
35
30
25
20
15
10
5
0
100.00%
90.00%
80.00%
70.00%
60.00%
50.00%
40.00%
30.00%
20.00%
10.00%
0.00%
Cummulative Percentage
81
82
83
84
85
86
87
88
89
90
91
92
93
94
95
96
97
98
99
100
More
Mean Measured Compaction
Frequency
Cumulative %

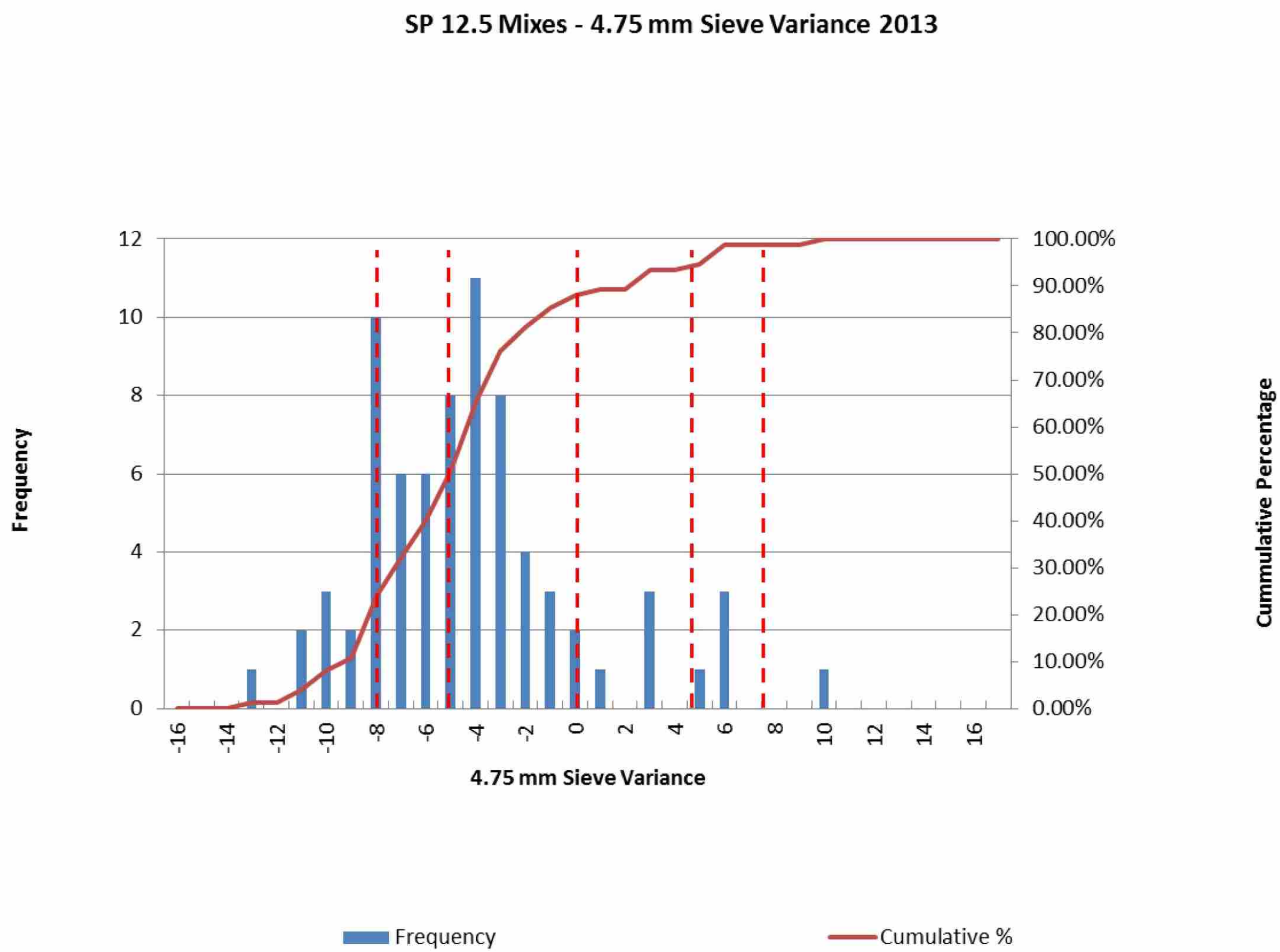
Maximum Measured Compaction 2012



Minimum Measured Compaction 2012



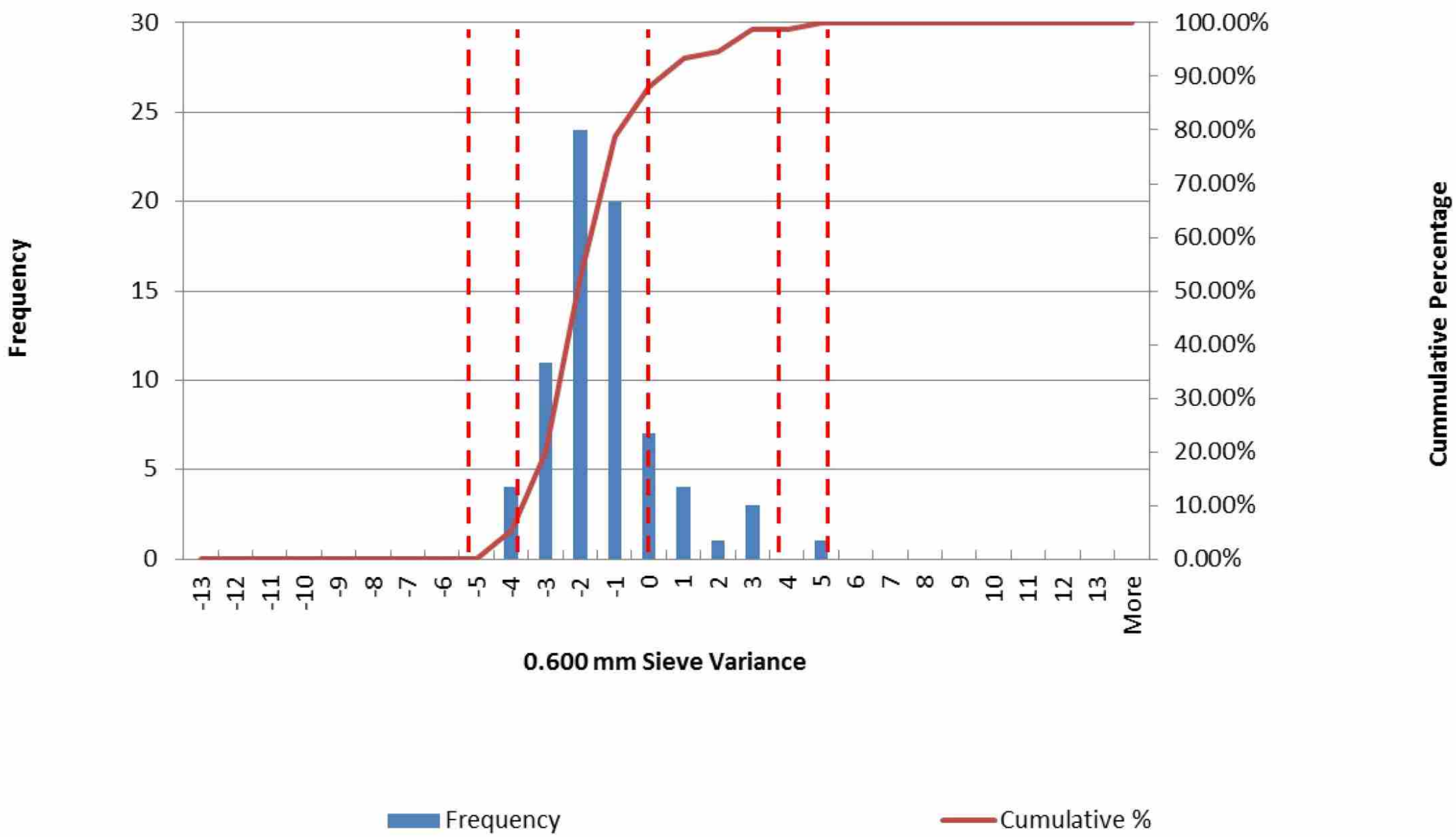
2013



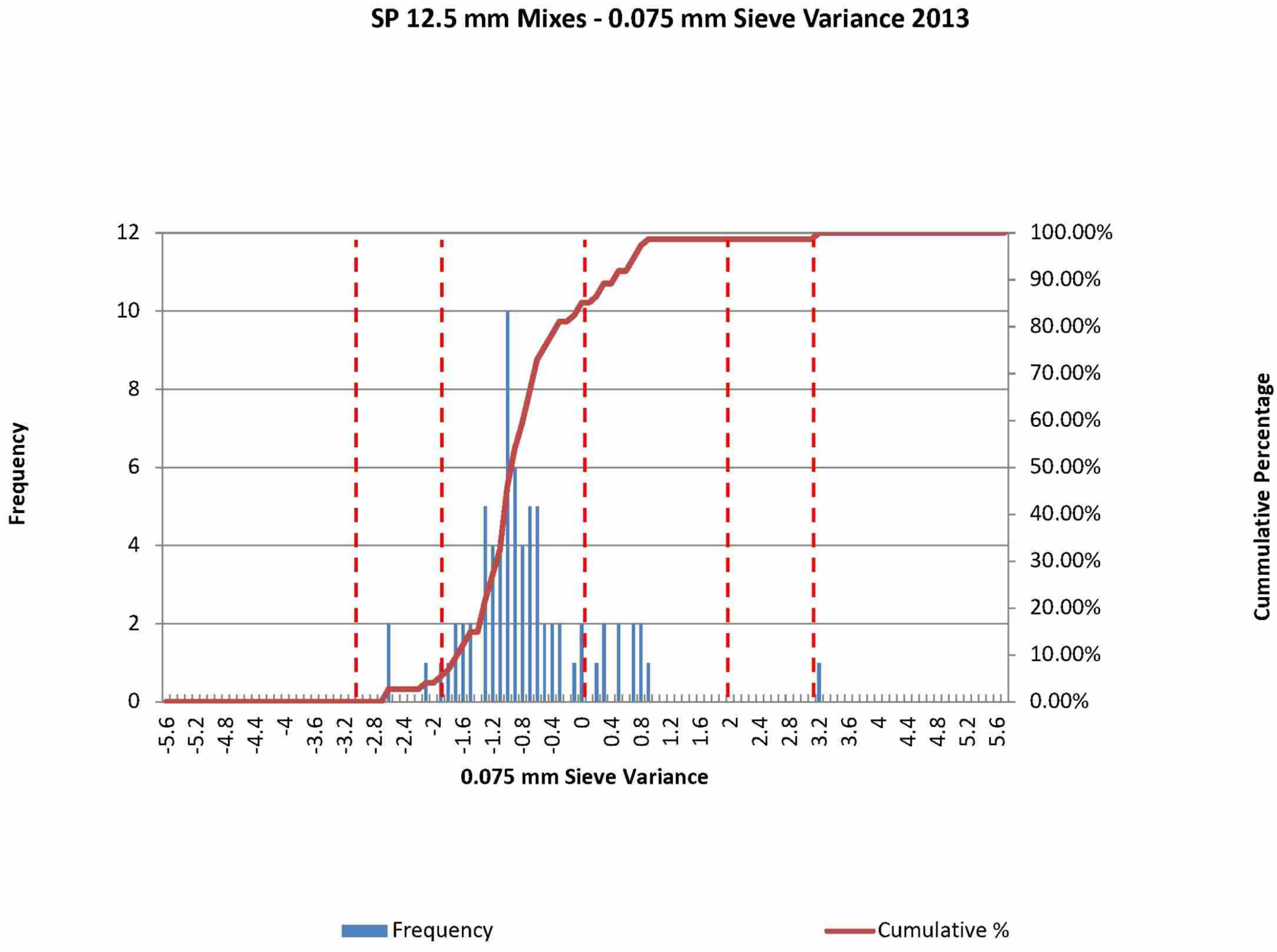
SP 12.5 Mixes - 4.75 mm Sieve Variance 2013
Frequency
Cummulative Percentage
12
10
8
6
4
2
0
100.00%
90.00%
80.00%
70.00%
60.00%
50.00%
40.00%
30.00%
20.00%
10.00%
0.00%
-16
-14
-12
-10
-8
-6
-4
-2
0
2
4
6
8
10
12
14
16
4.75 mm Sieve Variance
Frequency
Cumulative %

GOL0007504

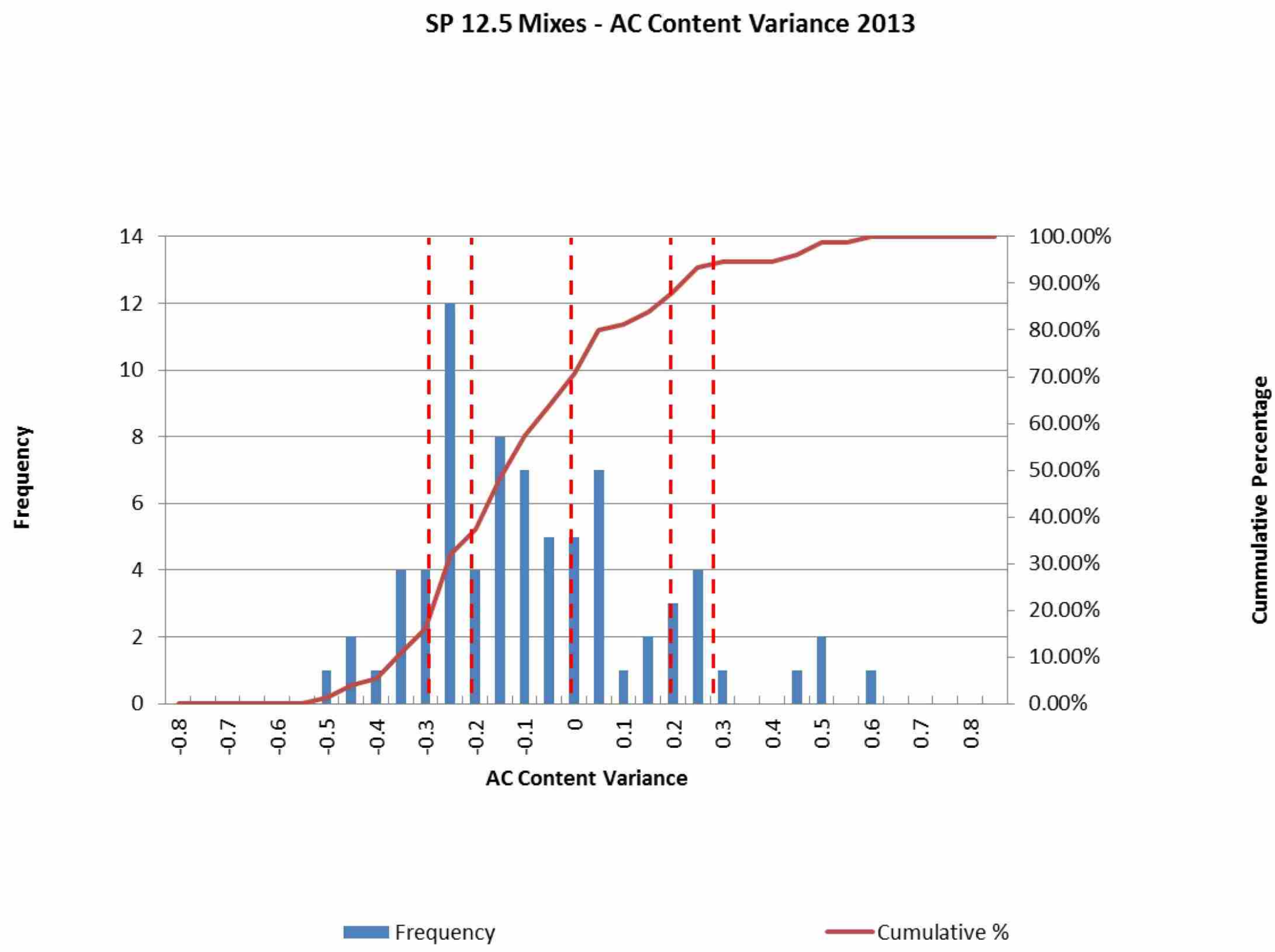
SP 12.5 Mixes - 0.600 mm Sieve Variance 2013



GOL0007504

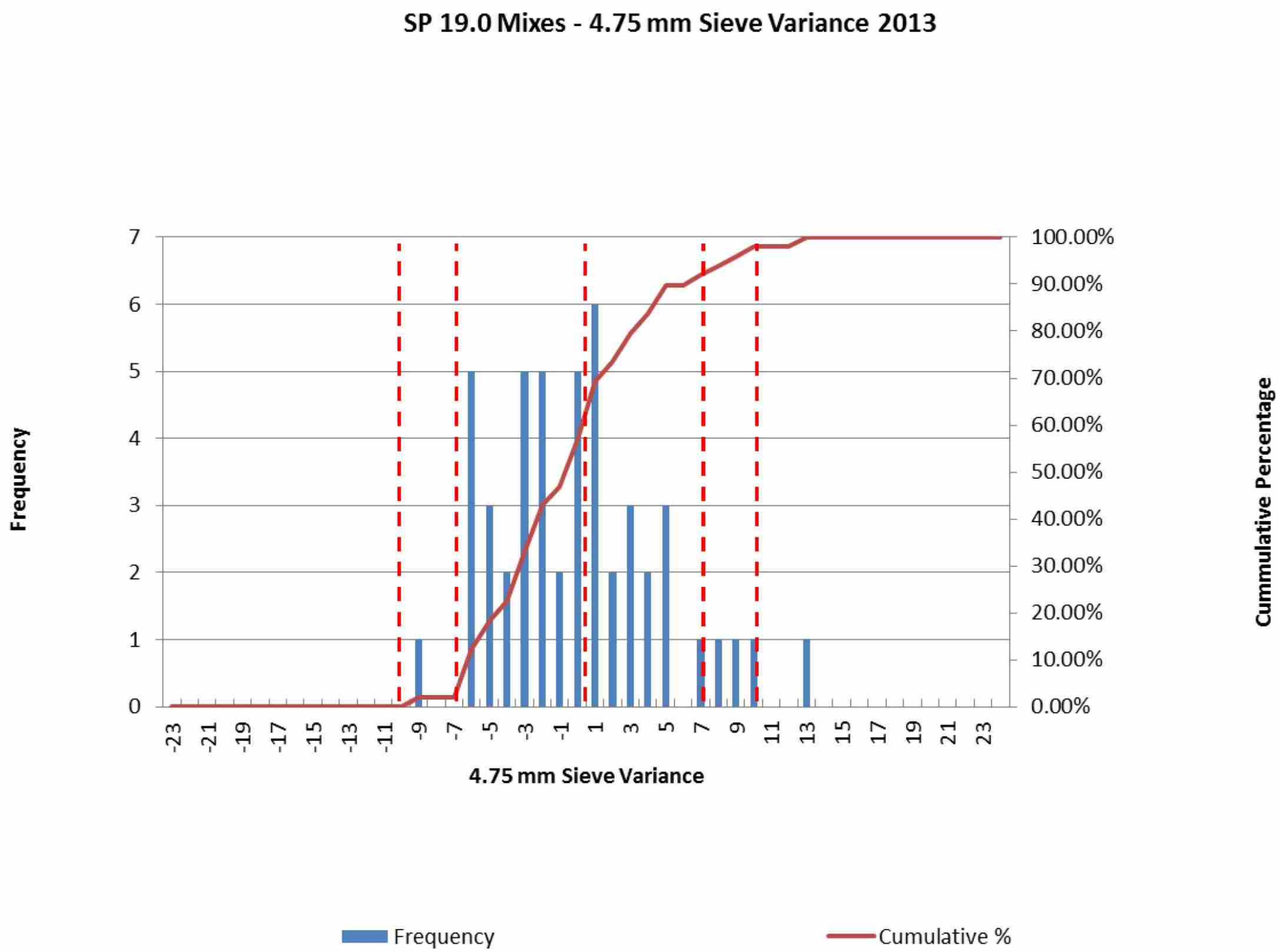

SP 12.5 mm Mixes - 0.075 mm Sieve Variance 2013
Frequency
Cummulative Percentage
0.075 mm Sieve Variance
0
2
4
6
8
10
12
0.00%
10.00%
20.00%
30.00%
40.00%
50.00%
60.00%
70.00%
80.00%
90.00%
100.00%
-5.6
-5.2
-4.8
-4.4
-4
-3.6
-3.2
-2.8
-2.4
-2
-1.6
-1.2
-0.8
-0.4
0
0.4
0.8
1.2
1.6
2
2.4
2.8
3.2
3.6
4
4.4
4.8
5.2
5.6
Frequency
Cumulative %

GOL0007504



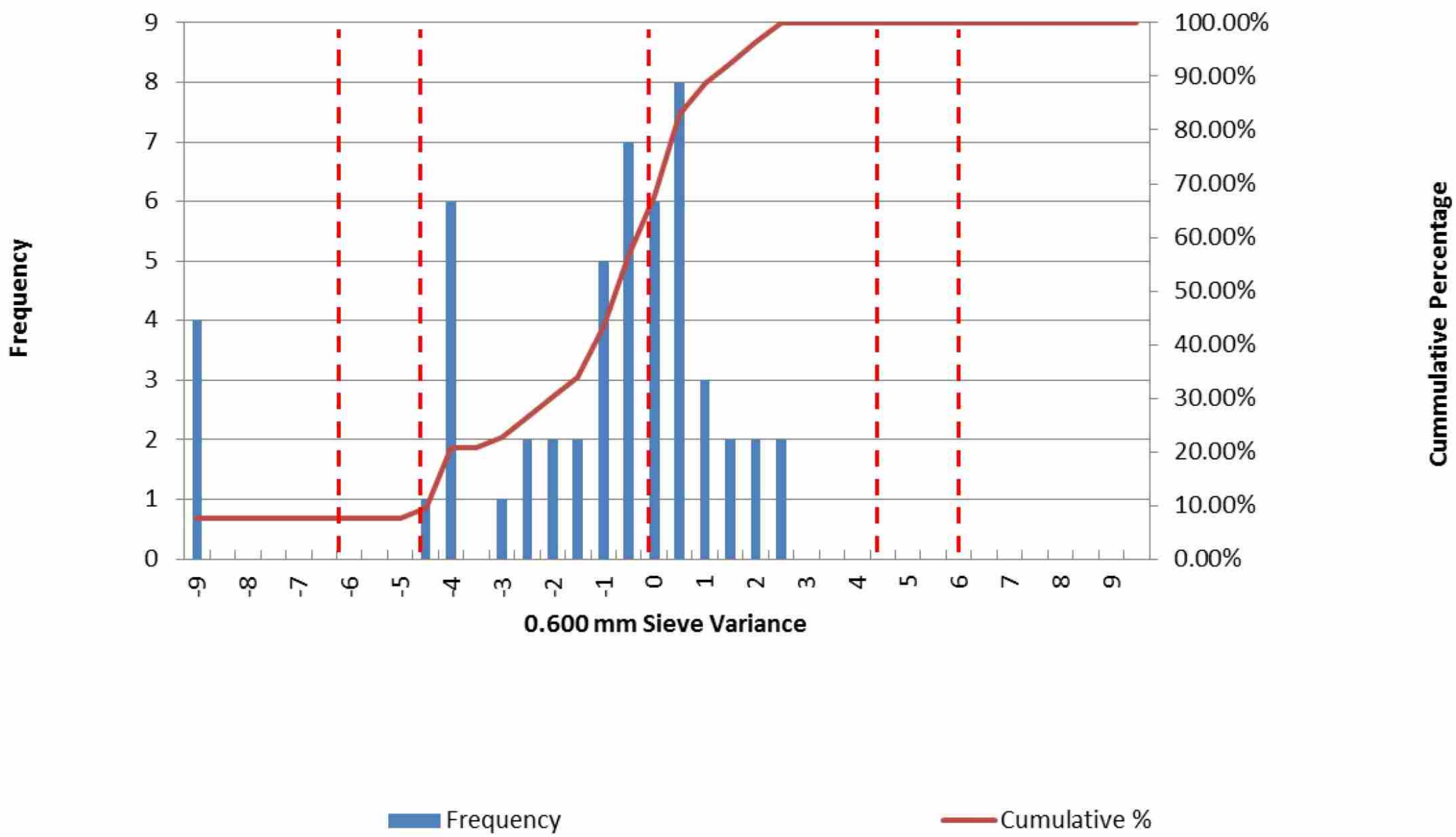
SP 12.5 Mixes - AC Content Variance 2013
Frequency
Cummulative Percentage
AC Content Variance
14
12
10
8
6
4
2
0
100.00%
90.00%
80.00%
70.00%
60.00%
50.00%
40.00%
30.00%
20.00%
10.00%
0.00%
-0.8
-0.7
-0.6
-0.5
-0.4
-0.3
-0.2
-0.1
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
Frequency
Cumulative %

GOL0007504


SP 19.0 Mixes - 4.75 mm Sieve Variance 2013
Frequency
Cummulative Percentage
4.75 mm Sieve Variance
0
1
2
3
4
5
6
7
0.00%
10.00%
20.00%
30.00%
40.00%
50.00%
60.00%
70.00%
80.00%
90.00%
100.00%
-23
-21
-19
-17
-15
-13
-11
-9
-7
-5
-3
-1
1
3
5
7
9
11
13
15
17
19
21
23
Frequency
Cumulative %

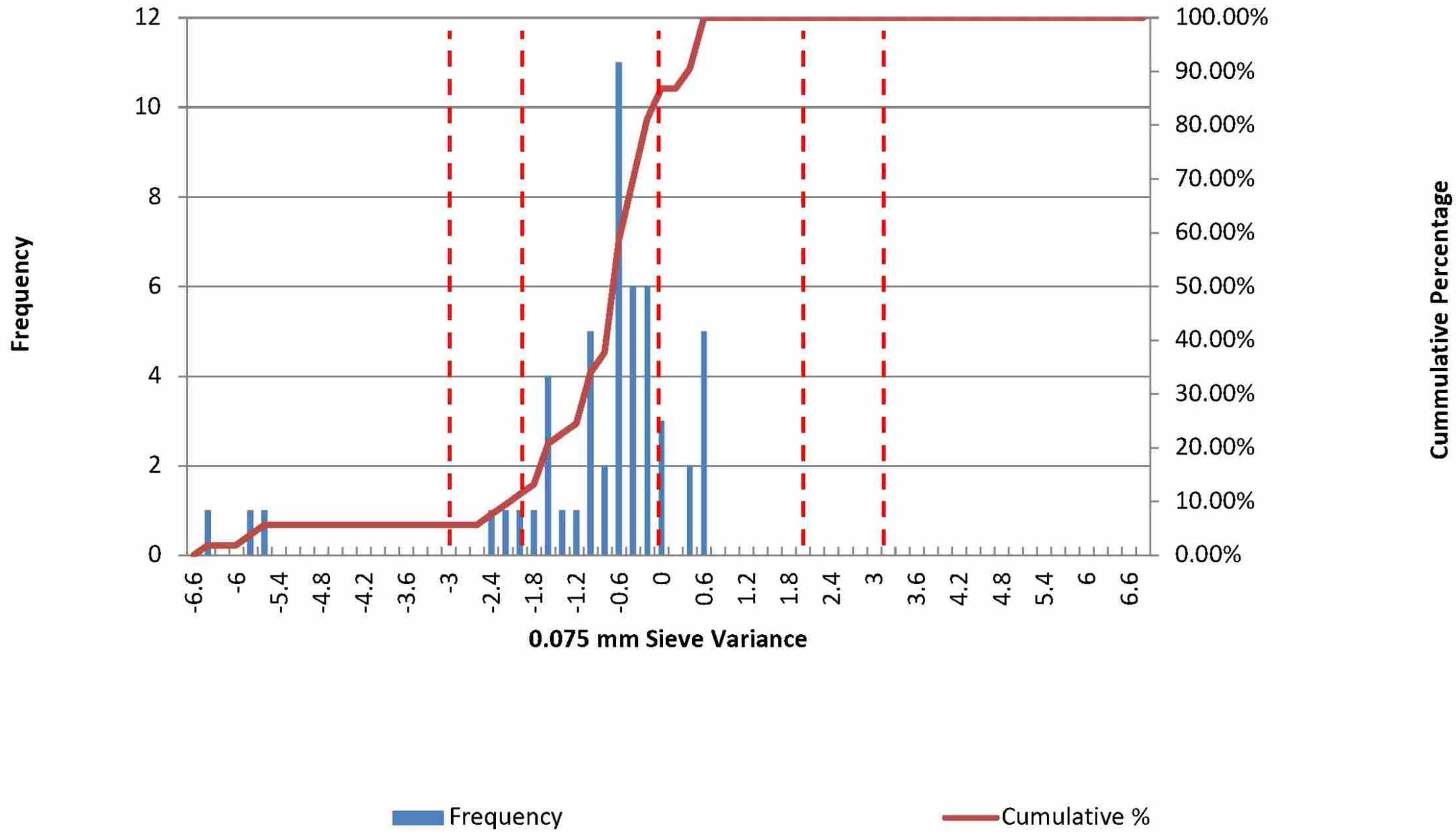
GOL0007504

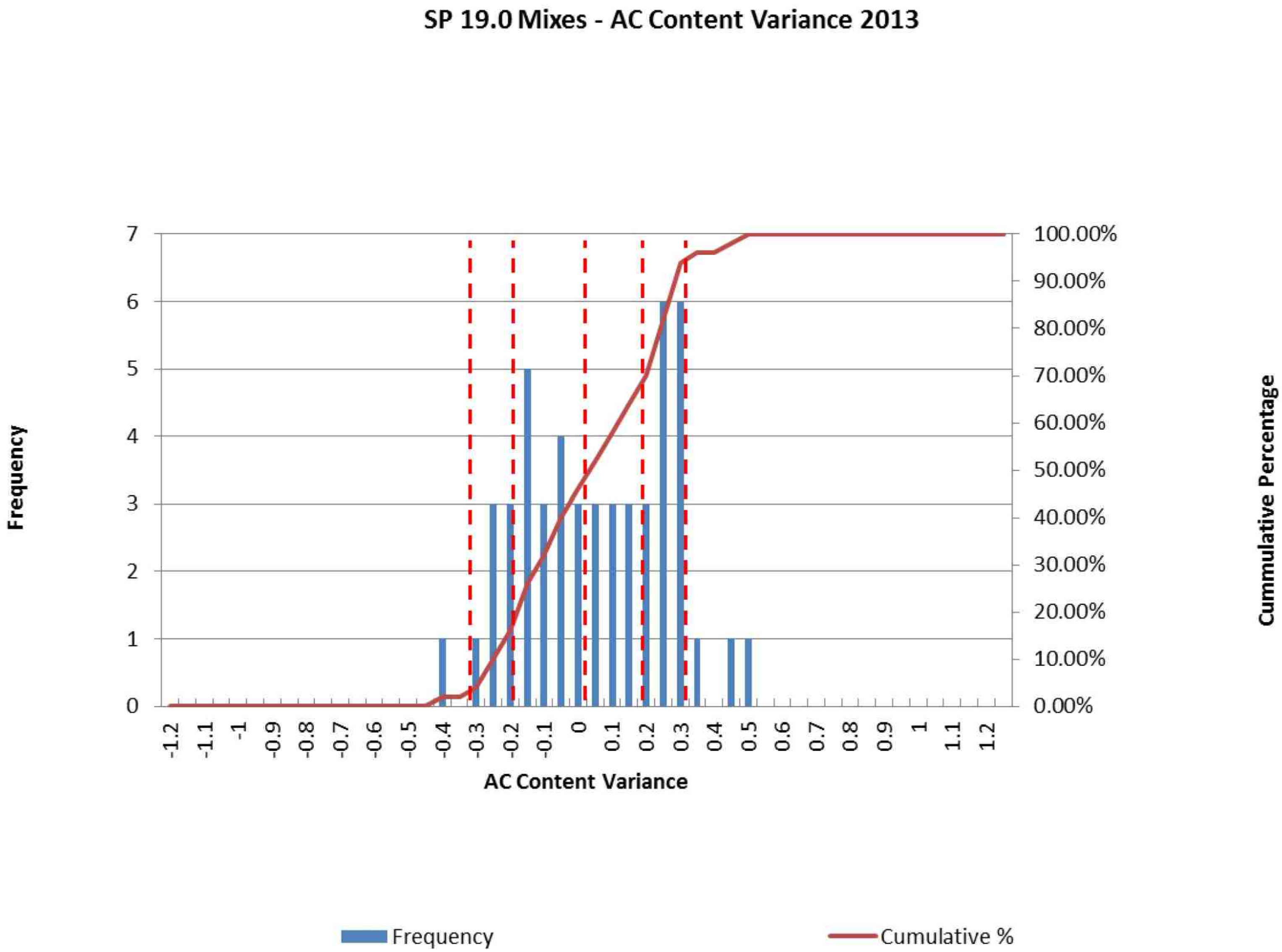
SP 19.0 mm Mixes - 0.600 mm Sieve Variance 2013



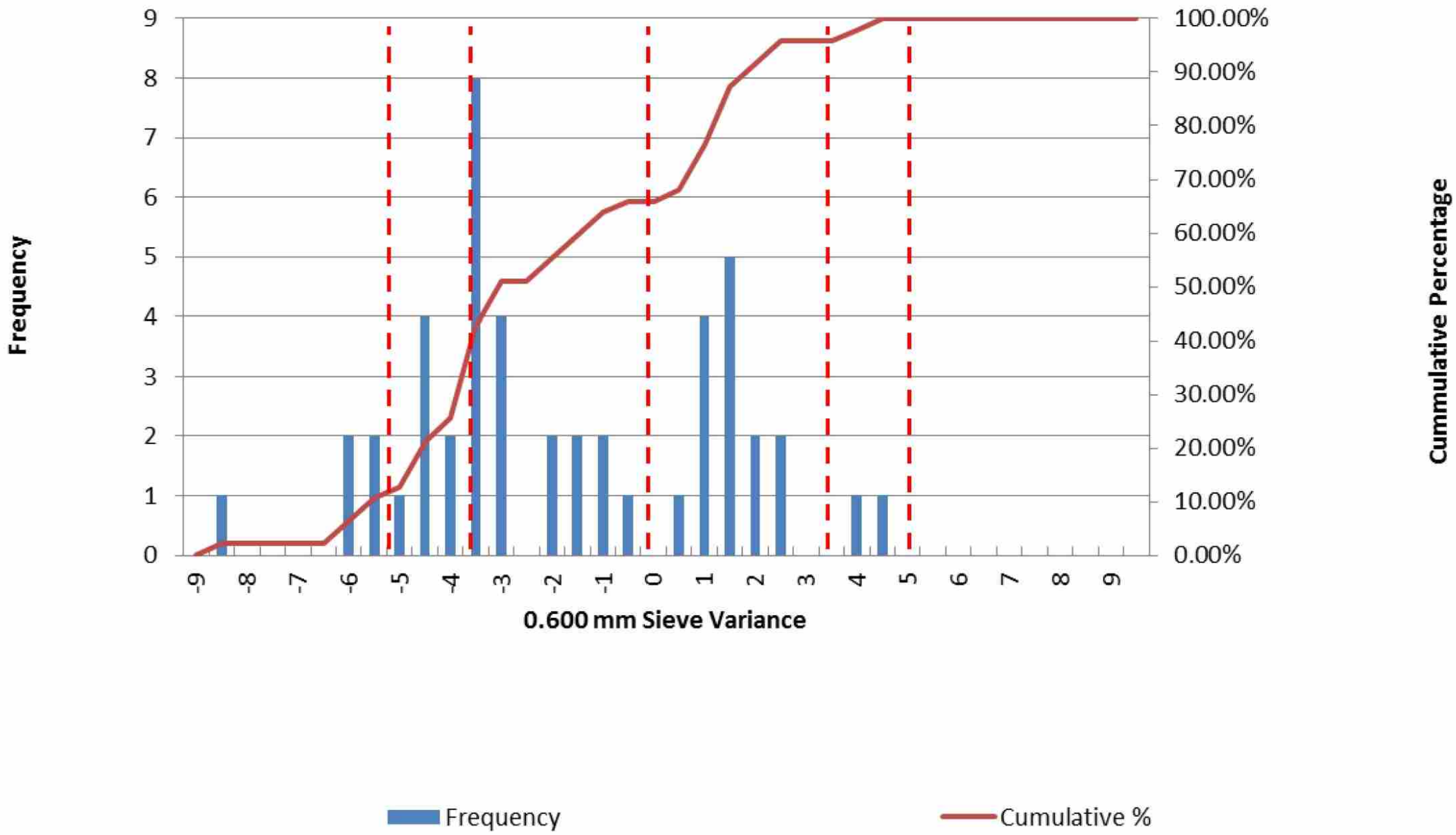
GOL0007504

SP 19.0 mm Mixes - 0.075 mm Sieve Variance 2013

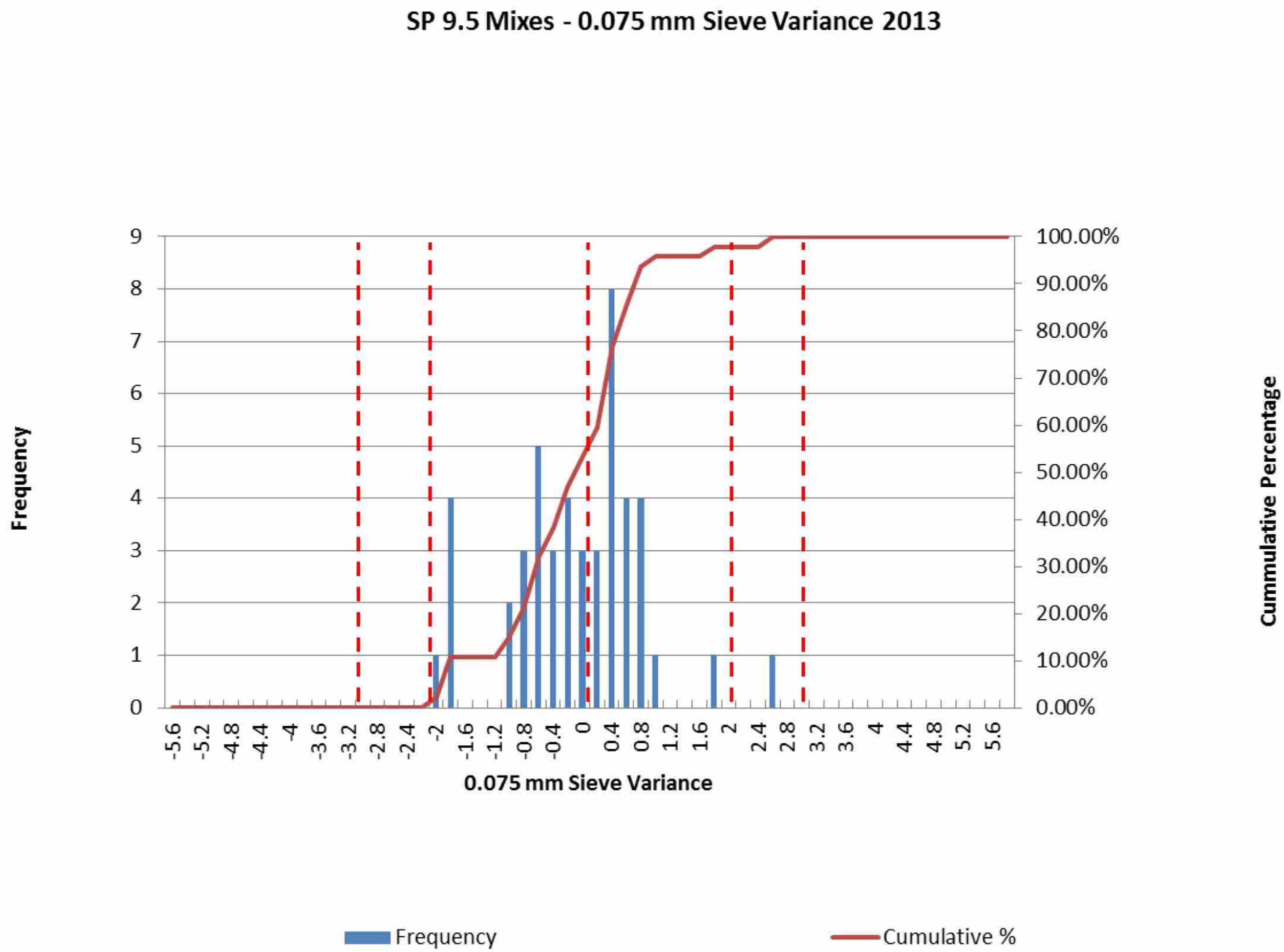



SP 19.0 Mixes - AC Content Variance 2013
Frequency
Cummulative Percentage
AC Content Variance
0
1
2
3
4
5
6
7
0.00%
10.00%
20.00%
30.00%
40.00%
50.00%
60.00%
70.00%
80.00%
90.00%
100.00%
-1.2
-1.1
-1
-0.9
-0.8
-0.7
-0.6
-0.5
-0.4
-0.3
-0.2
-0.1
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
1.1
1.2
Frequency
Cumulative %

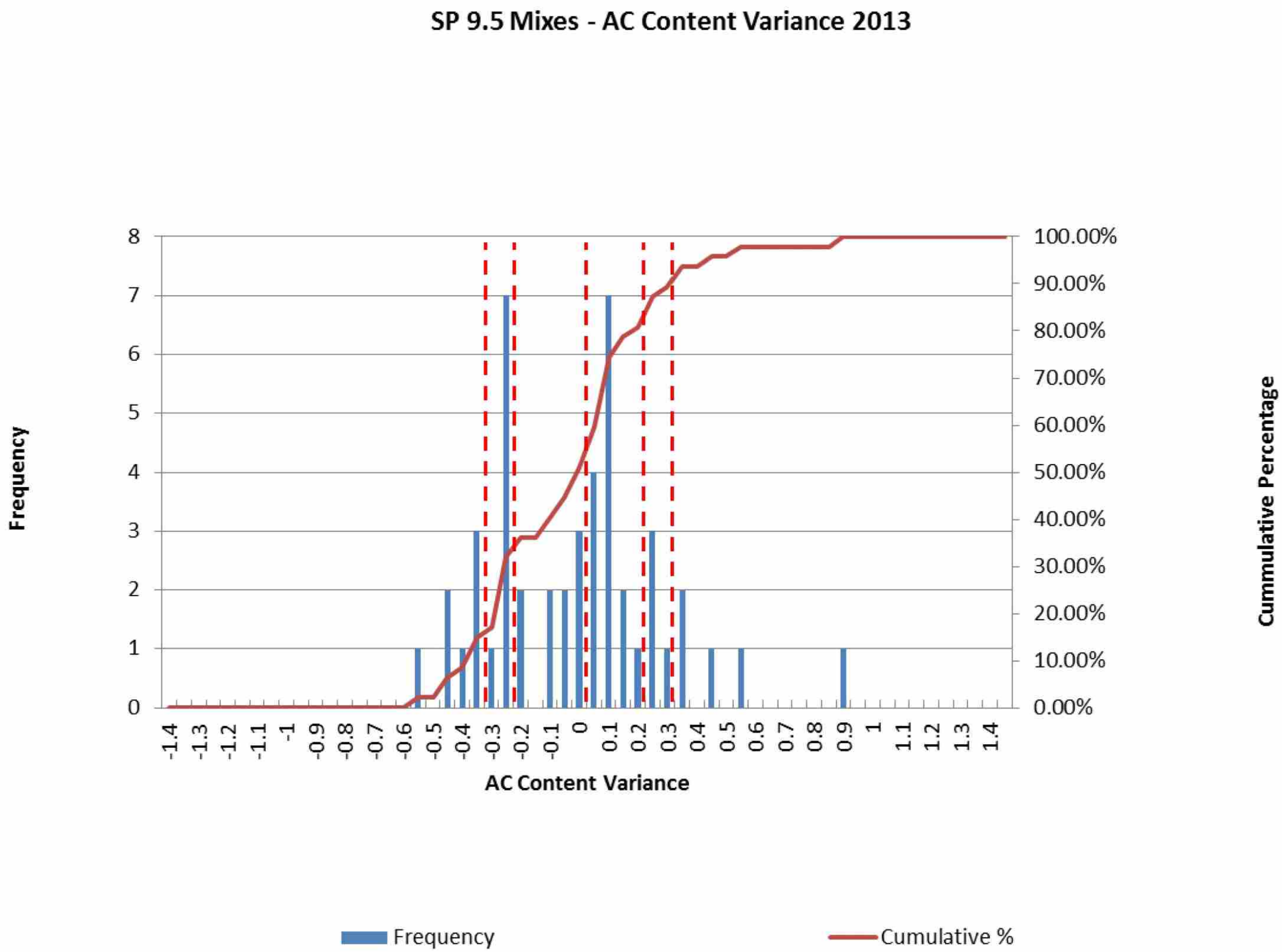
SP 9.5 Mixes - 0.600 mm Sieve Variance 2013

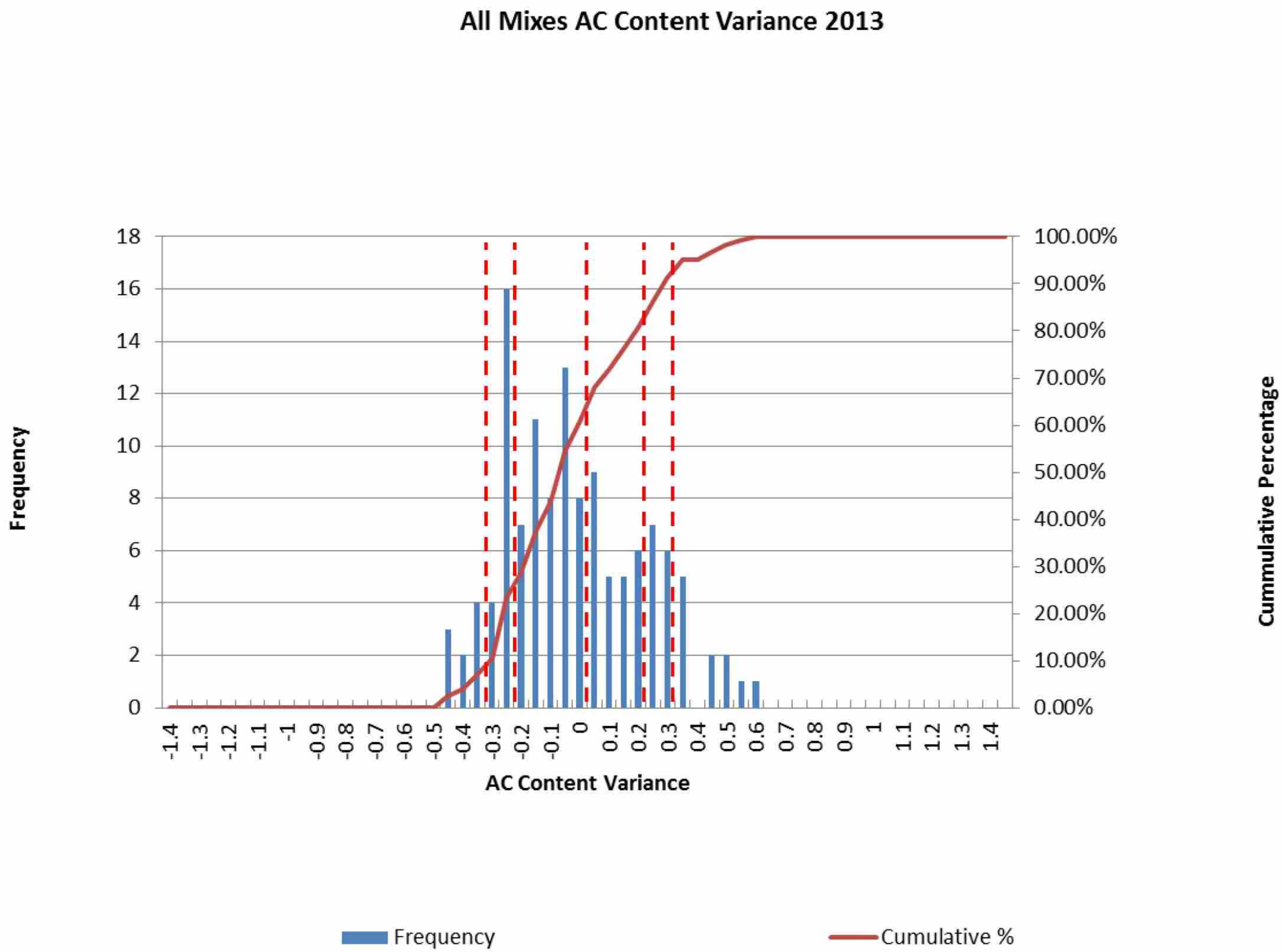


GOL0007504

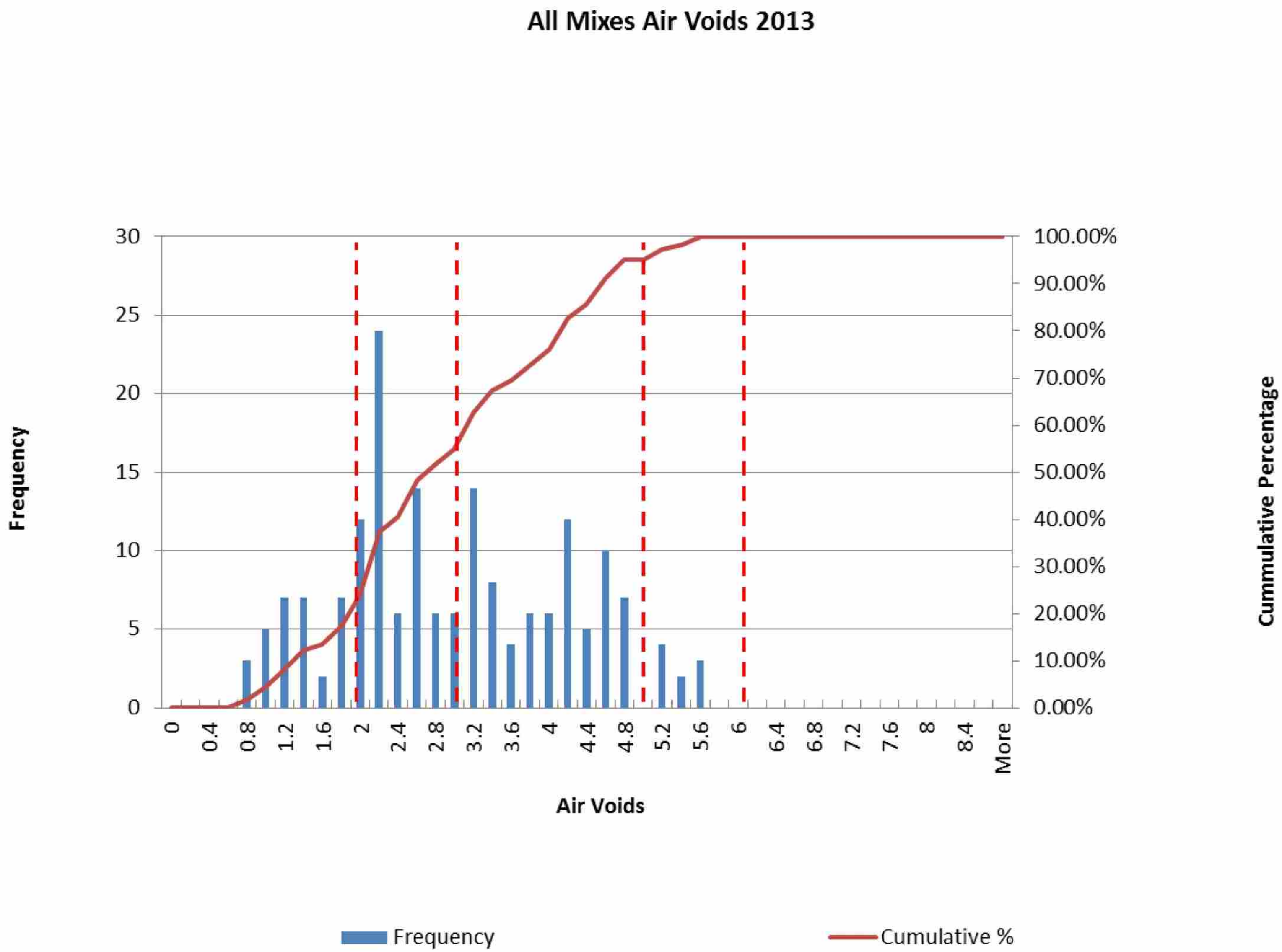


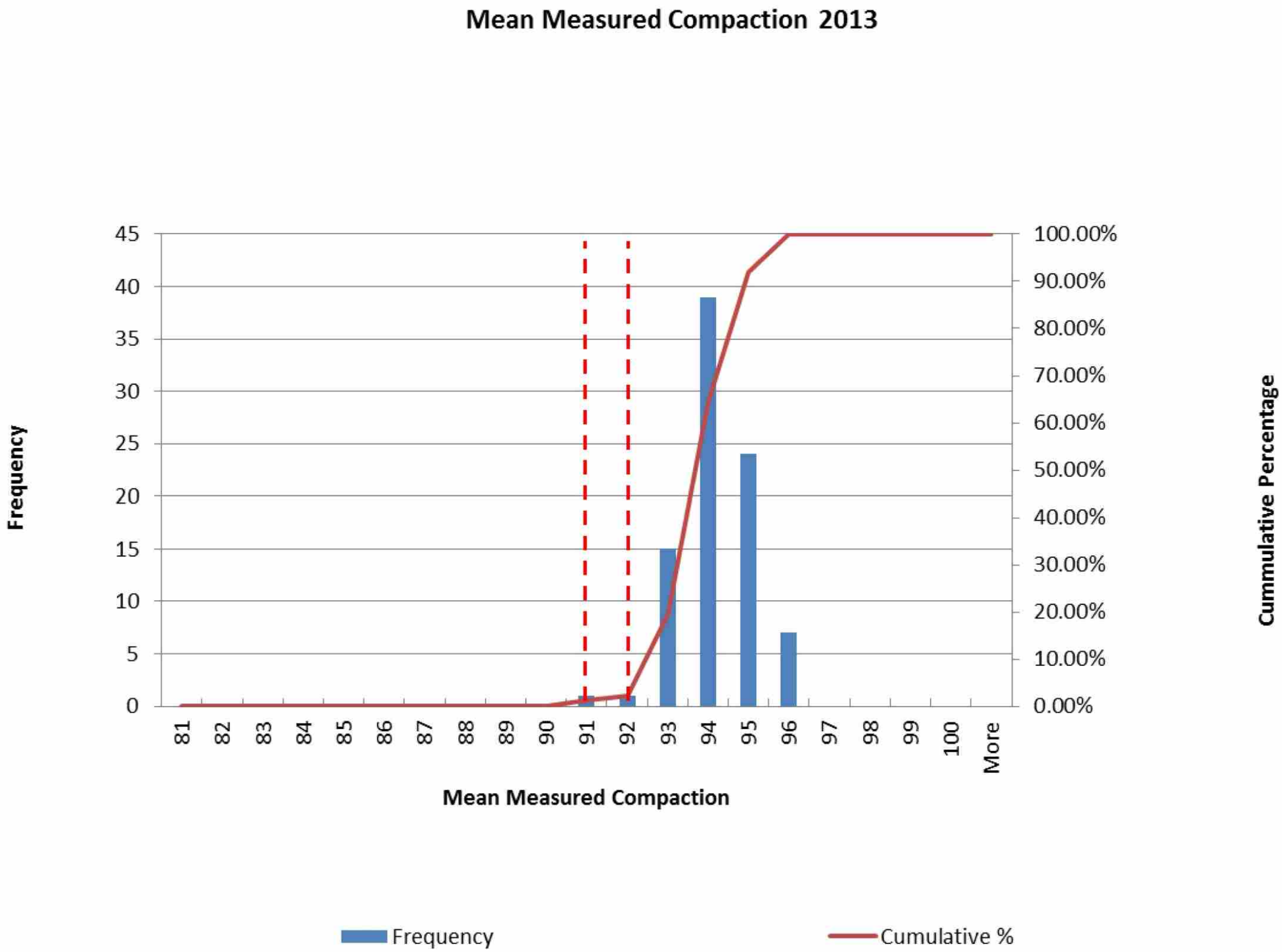
SP 9.5 Mixes - 0.075 mm Sieve Variance 2013
Frequency
Cummulative Percentage
0.075 mm Sieve Variance
0
1
2
3
4
5
6
7
8
9
0.00%
10.00%
20.00%
30.00%
40.00%
50.00%
60.00%
70.00%
80.00%
90.00%
100.00%
-5.6
-5.2
-4.8
-4.4
-4
-3.6
-3.2
-2.8
-2.4
-2
-1.6
-1.2
-0.8
-0.4
0
0.4
0.8
1.2
1.6
2
2.4
2.8
3.2
3.6
4
4.4
4.8
5.2
5.6
Frequency
Cumulative %


SP 9.5 Mixes - AC Content Variance 2013
Frequency
0
1
2
3
4
5
6
7
8
Cummulative Percentage
0.00%
10.00%
20.00%
30.00%
40.00%
50.00%
60.00%
70.00%
80.00%
90.00%
100.00%
-1.4 -1.3 -1.2 -1.1 -1 -0.9 -0.8 -0.7 -0.6 -0.5 -0.4 -0.3 -0.2 -0.1 0 0.1 0.2 0.3 0.4 0.5 0.6 0.7 0.8 0.9 1 1.1 1.2 1.3 1.4
AC Content Variance
Frequency
Cumulative %


All Mixes AC Content Variance 2013
Frequency
Cummulative Percentage
18
16
14
12
10
8
6
4
2
0
100.00%
90.00%
80.00%
70.00%
60.00%
50.00%
40.00%
30.00%
20.00%
10.00%
0.00%
-1.4
-1.3
-1.2
-1.1
-1
-0.9
-0.8
-0.7
-0.6
-0.5
-0.4
-0.3
-0.2
-0.1
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
1.1
1.2
1.3
1.4
AC Content Variance
Frequency
Cumulative %

GOL0007504

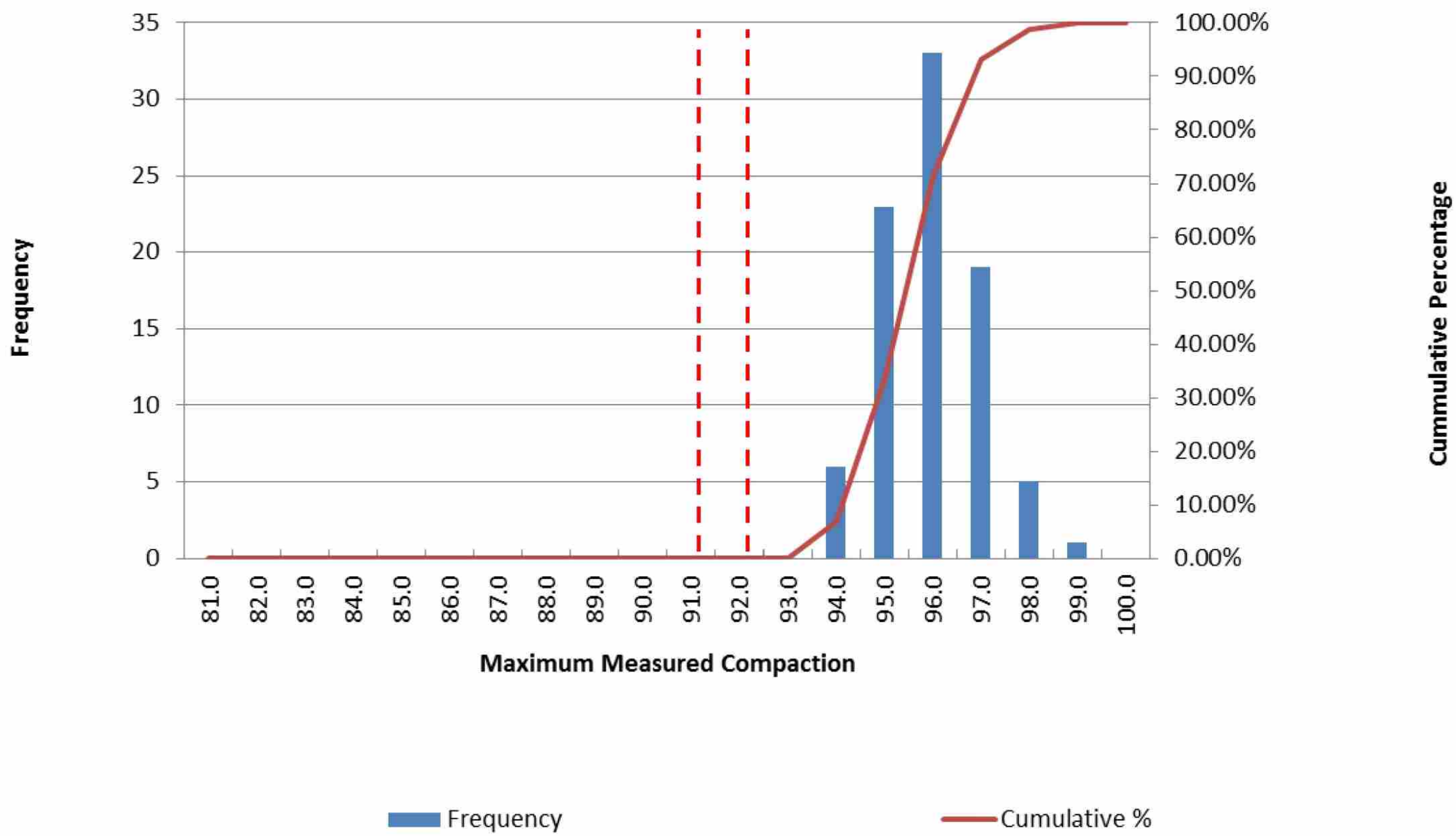

All Mixes Air Voids 2013
Frequency
Cummulative Percentage
Air Voids
0
5
10
15
20
25
30
0.00%
10.00%
20.00%
30.00%
40.00%
50.00%
60.00%
70.00%
80.00%
90.00%
100.00%
0
0.4
0.8
1.2
1.6
2
2.4
2.8
3.2
3.6
4
4.4
4.8
5.2
5.6
6
6.4
6.8
7.2
7.6
8
8.4
More
Frequency
Cumulative %



Mean Measured Compaction 2013
Frequency
45
40
35
30
25
20
15
10
5
0
100.00%
90.00%
80.00%
70.00%
60.00%
50.00%
40.00%
30.00%
20.00%
10.00%
0.00%
Cummulative Percentage
81
82
83
84
85
86
87
88
89
90
91
92
93
94
95
96
97
98
99
100
More
Mean Measured Compaction
Frequency
Cumulative %

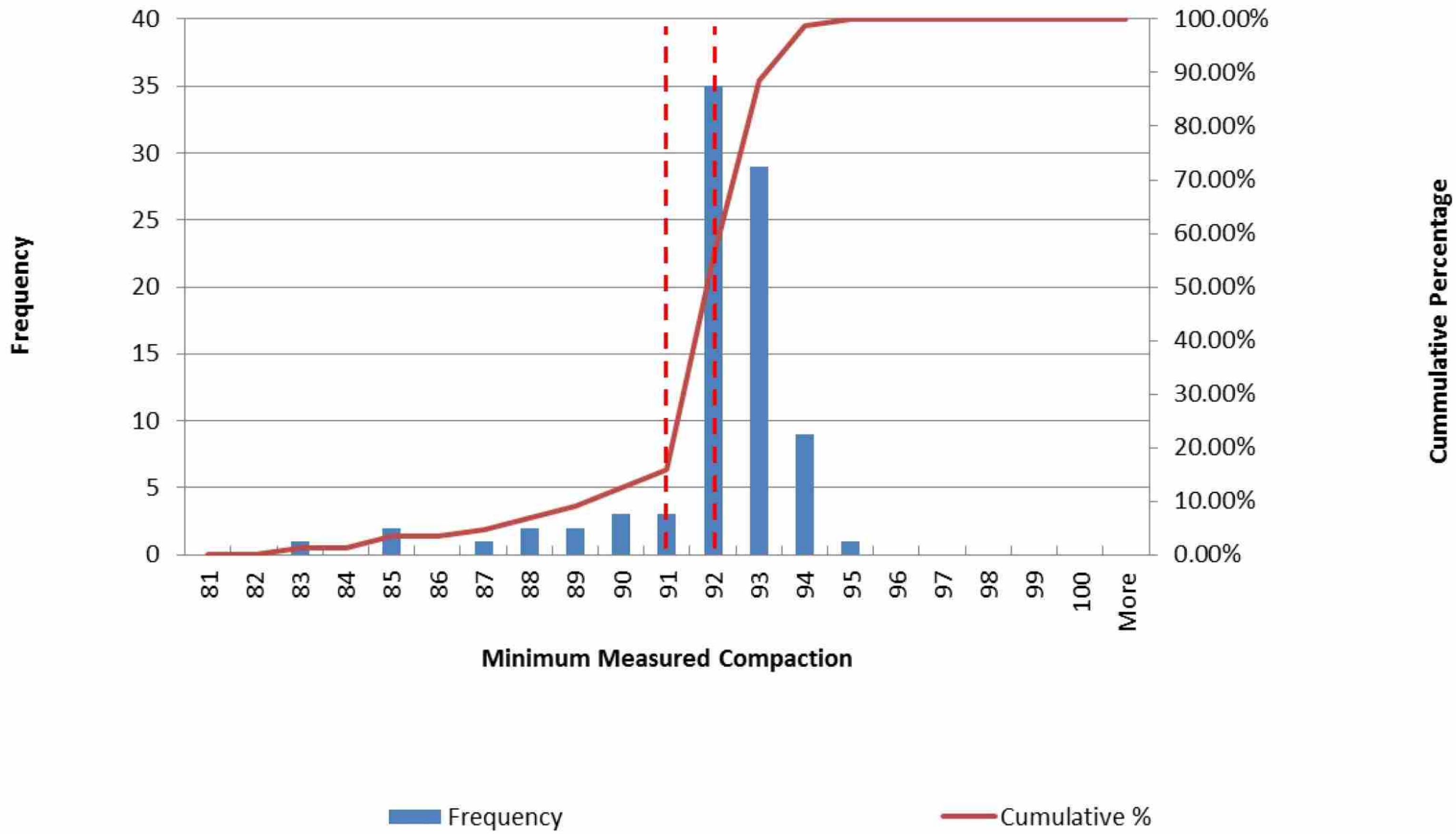
GOL0007504

Maximum Measured Compaction 2013



GOL0007504

Minimum Measured Compaction 2013



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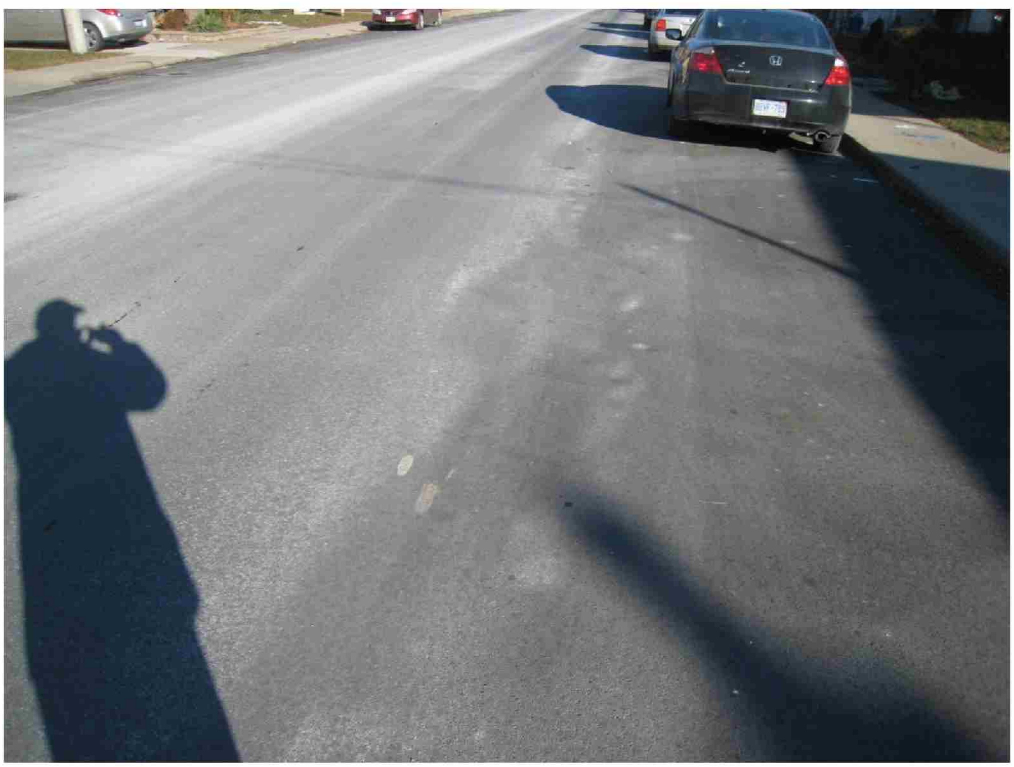
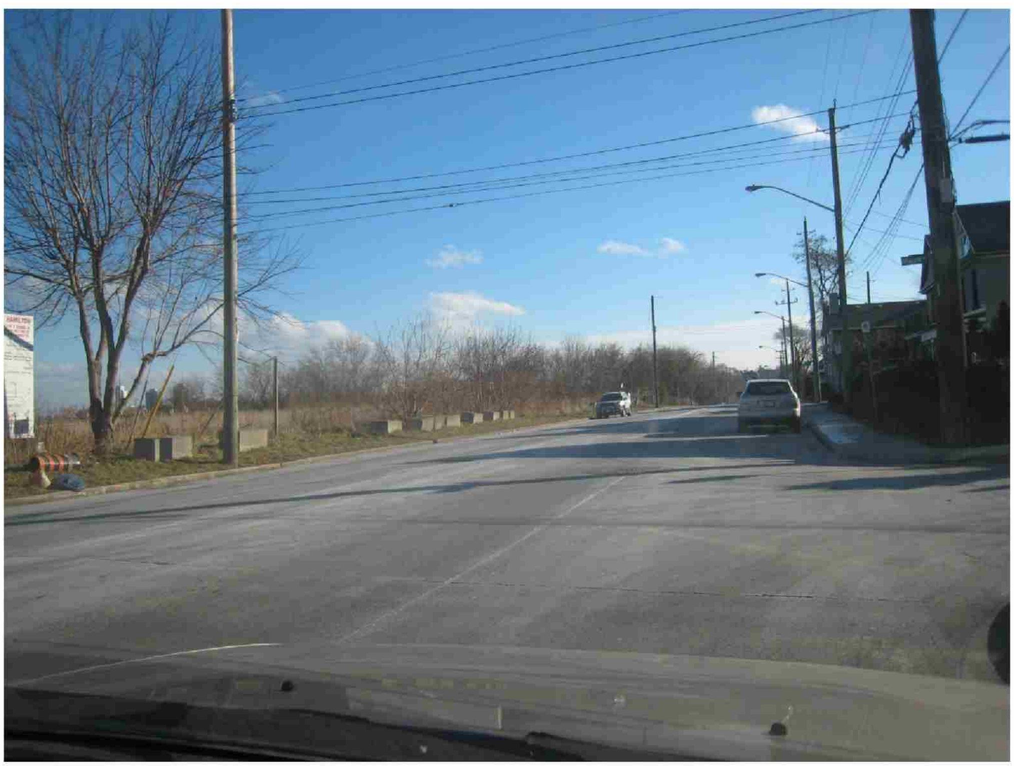
APPENDIX B

Photographs from 2013 Visual Condition Inspection

Barton Street Paved 2012

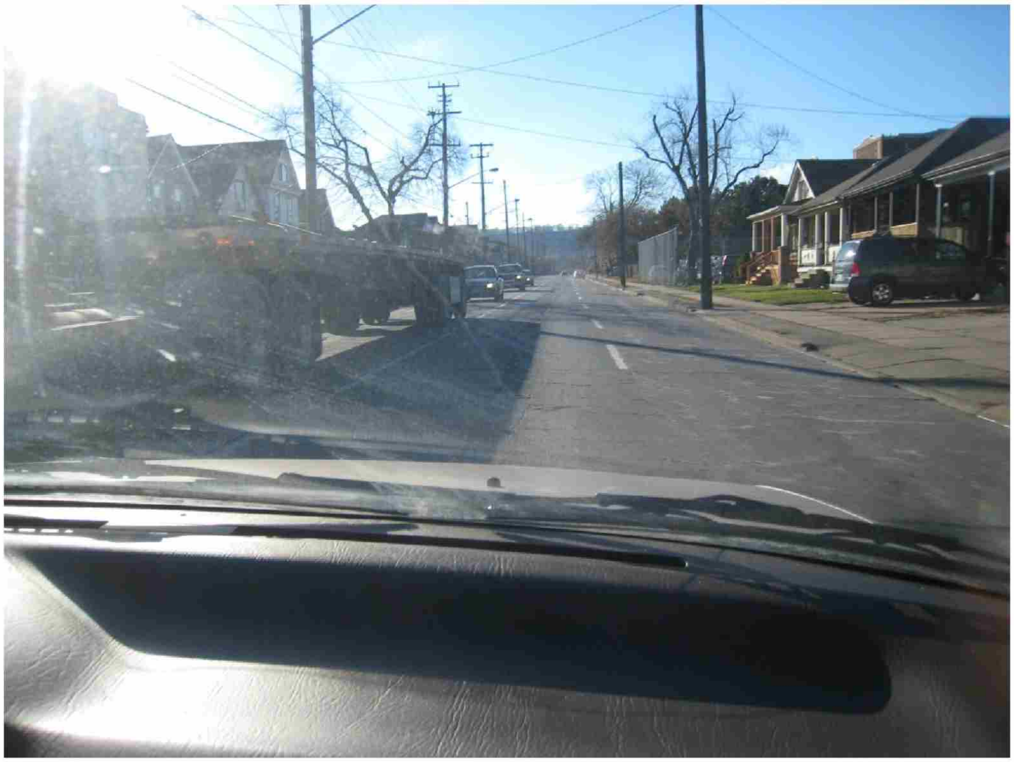




Huron Street Paved in 2012

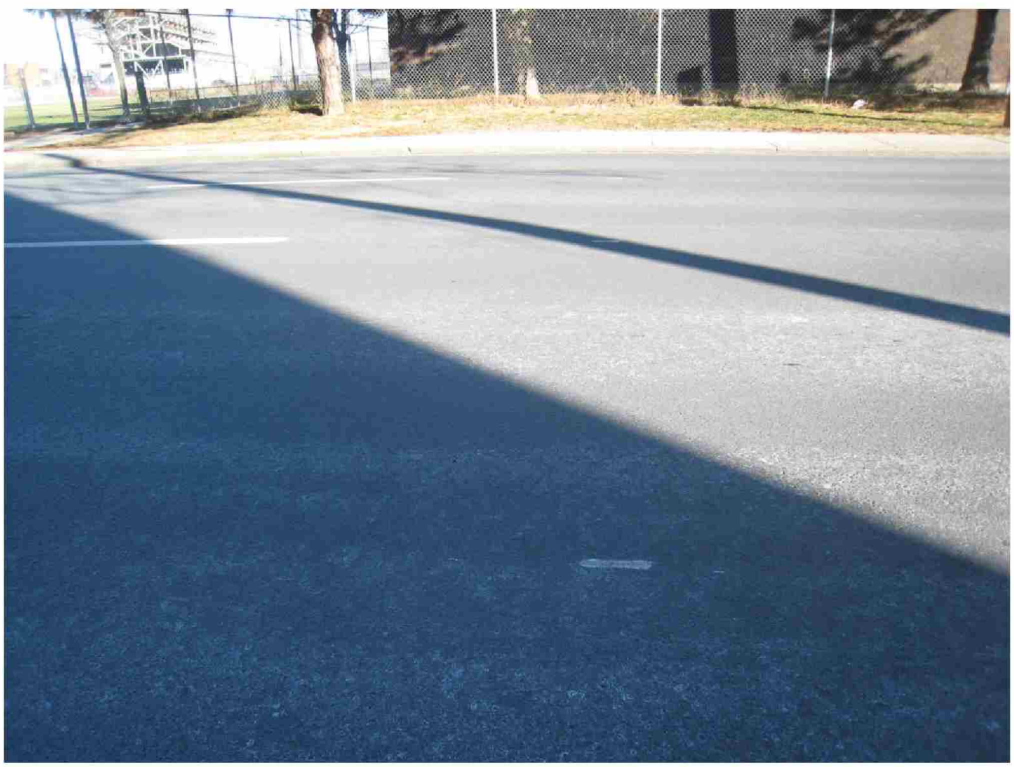
GOL0007504

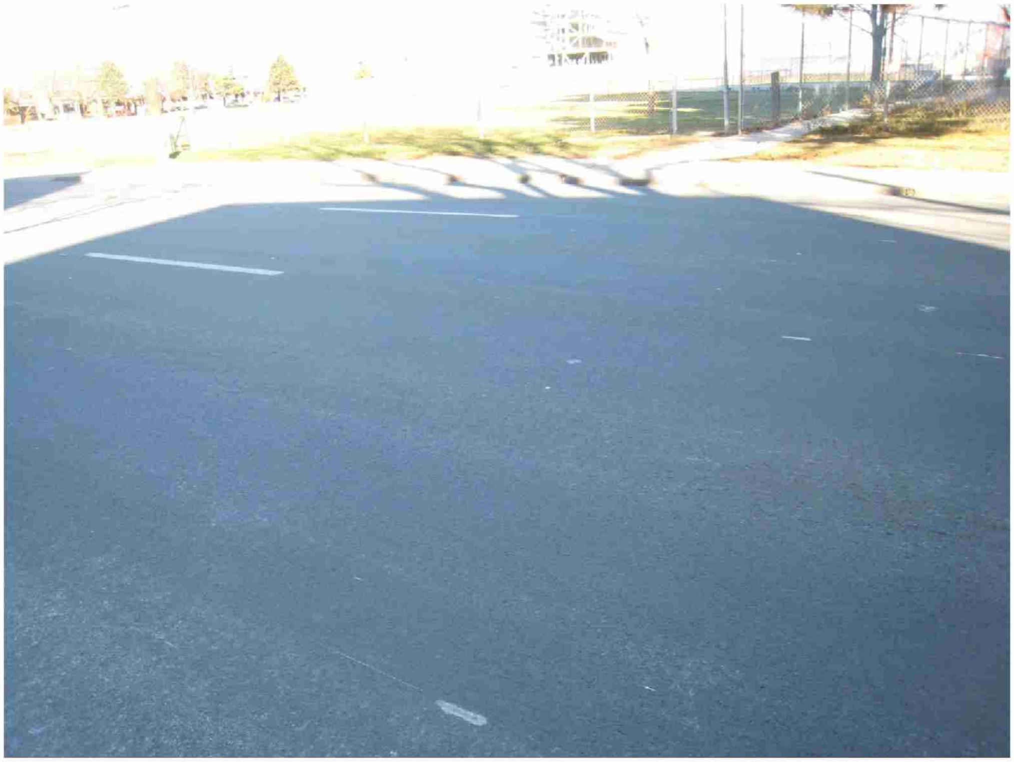
Typical Condition of Pavement Containing Steel Slag



GOL0007504

King Street Paved in 2013



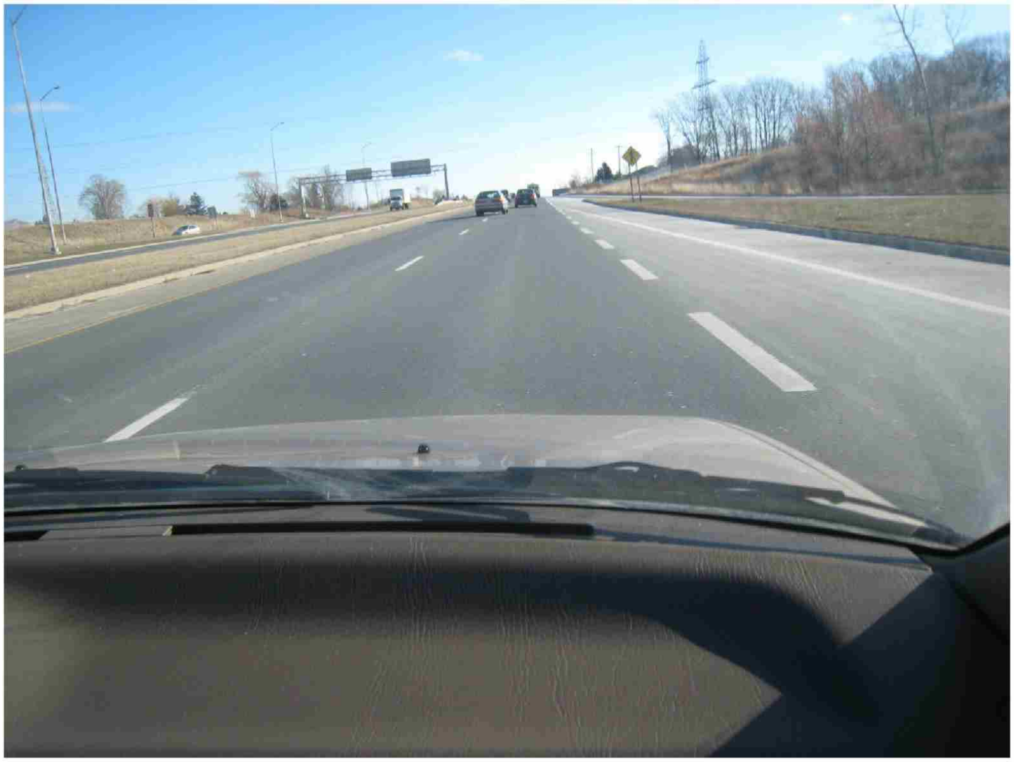


Mohawk Road Paved in 2013 with Fibers

GOL0007504

Mohawk Road Paved in 2013 without Fibers

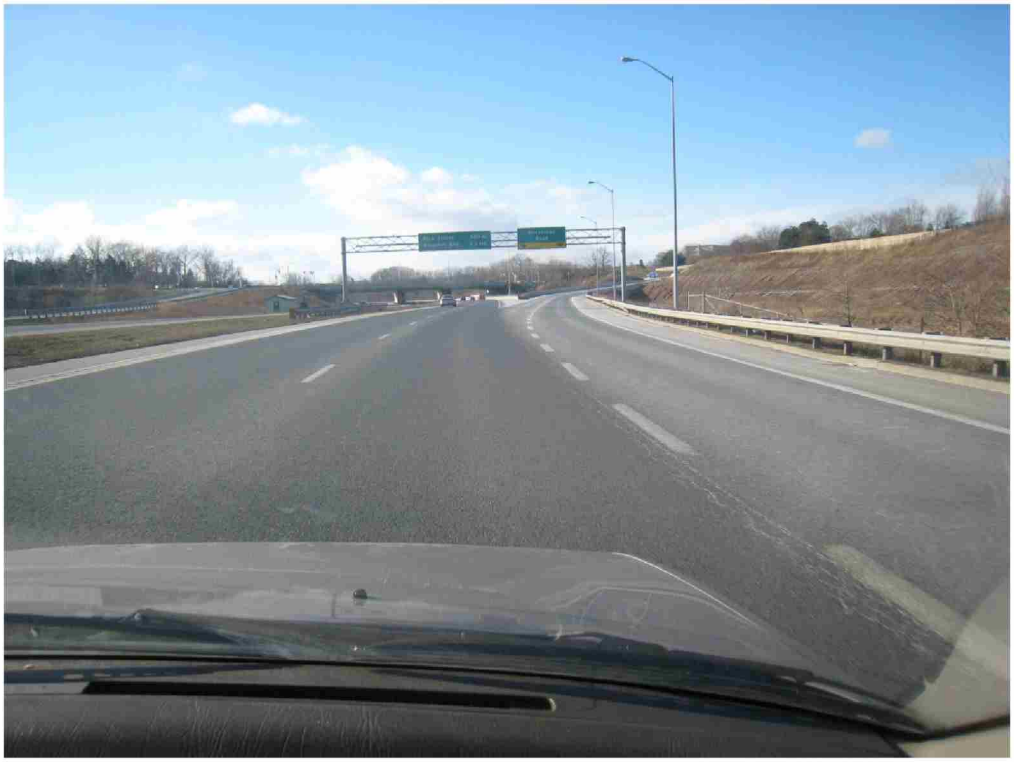
LINC Paved in 2010

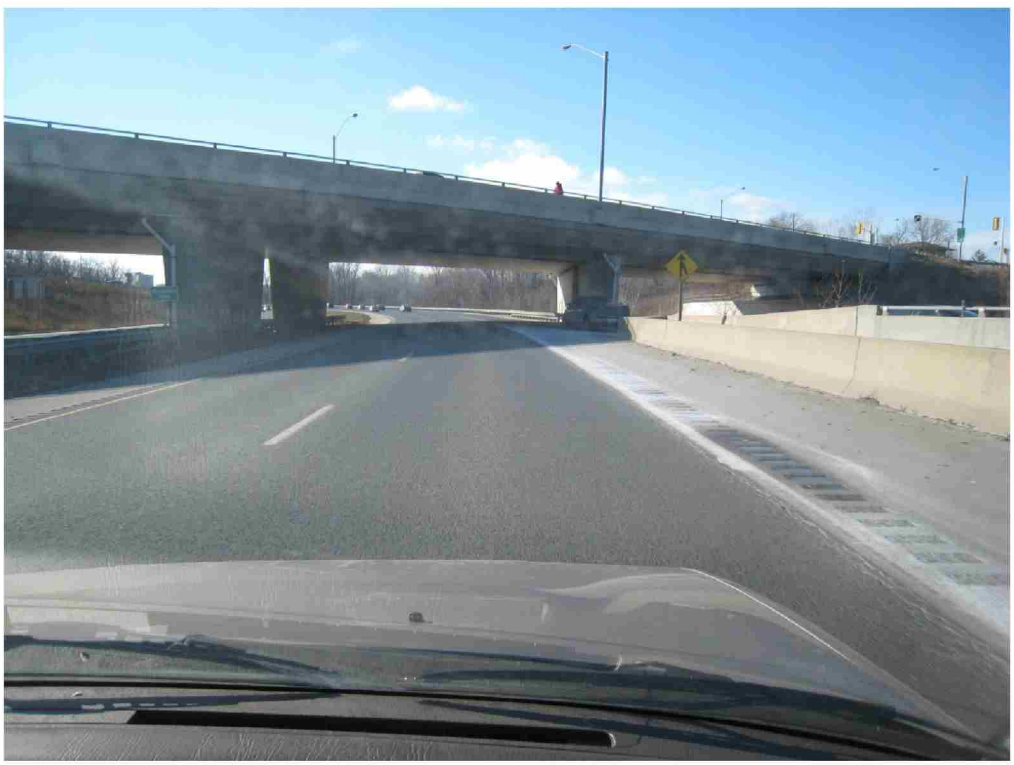


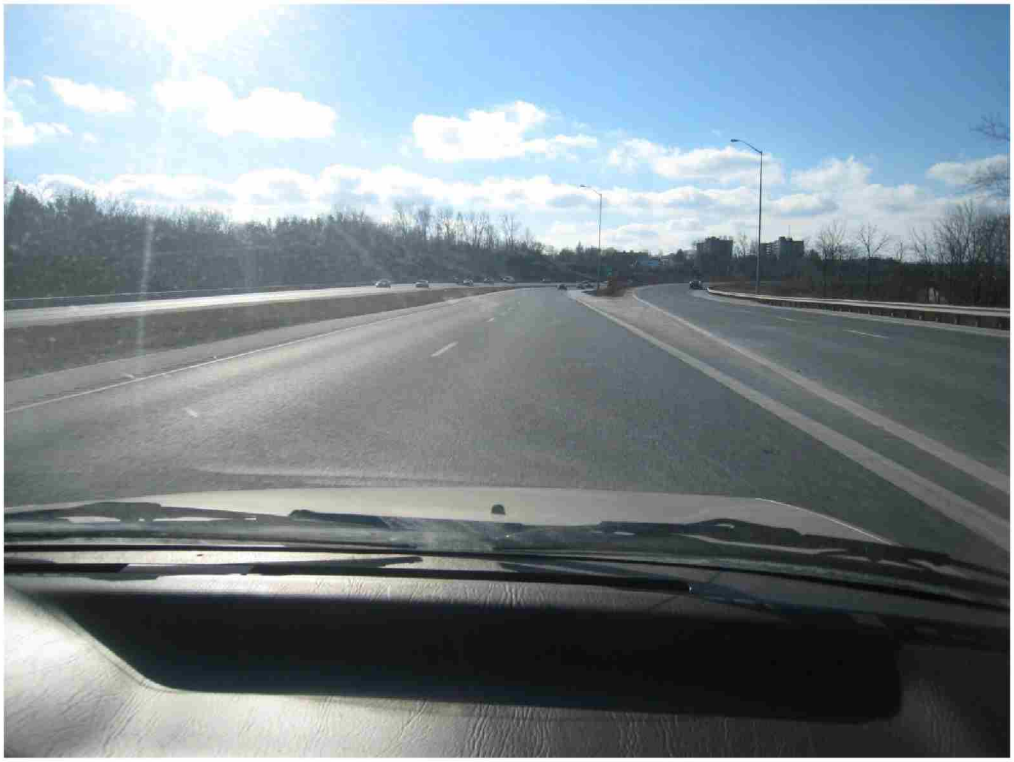

40

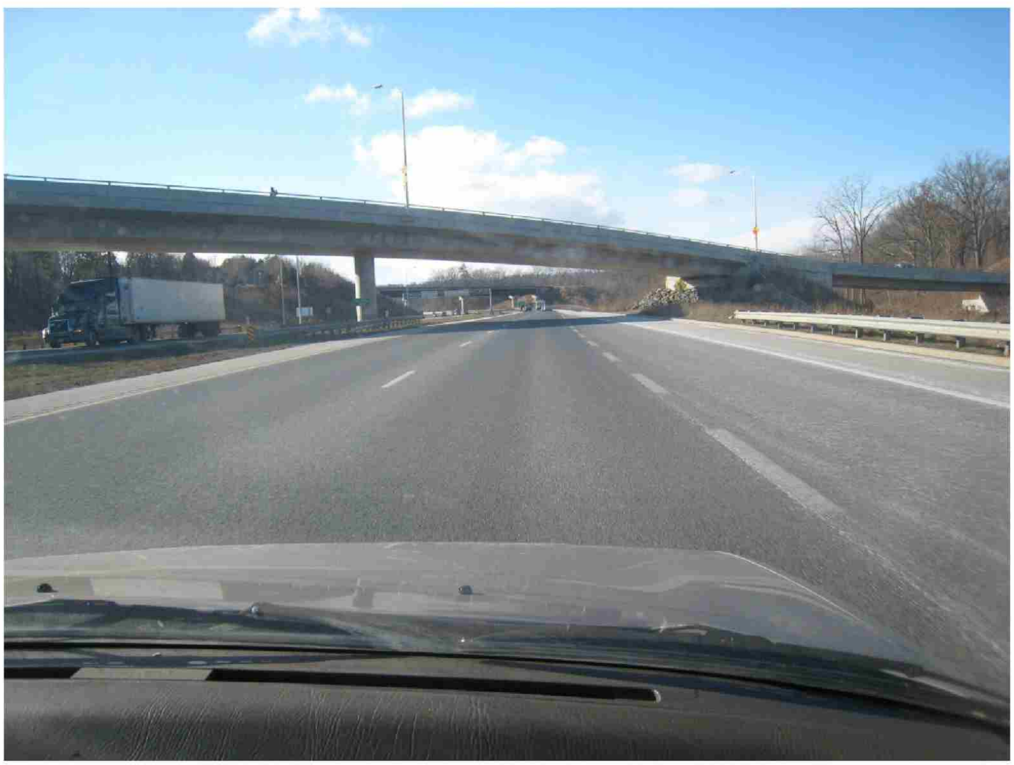
RHVP Paved in 2007

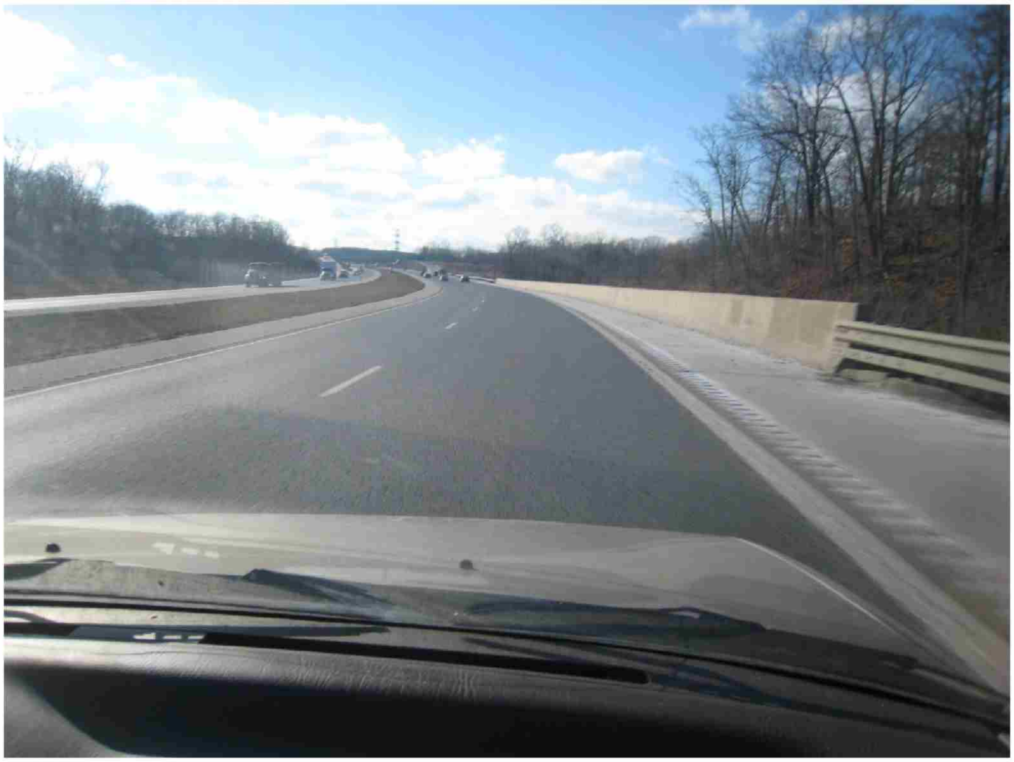
GOL0007504



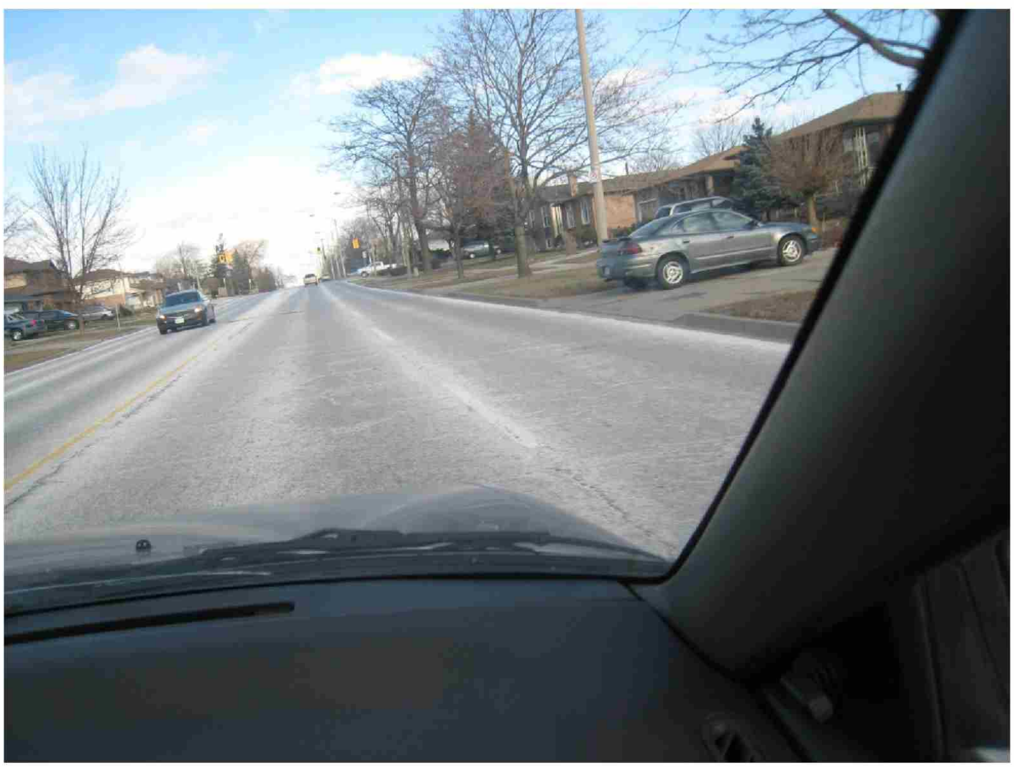








Sherman Avenue Paved with SMA containing MSM



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APPENDIX C

Updated Pavement Structural Design Matrix

Road Type	Pavement Type	Subgrade Soil Type	Material Type and Thickness			
			Surface Course	Binder Course	Base Course	Granular Subbase or Granular Base
Local	Flexible	Clayey Silt	40 mm SP 9.5 Cat C with PG 58-28	80 mm SP 19.0 Cat C with PG 58-28	150 mm Granular A (500 mm[1])	400 mm of Granular B, Type II
		Clay	40 mm SP 9.5 Cat C with PG 58-28	80 mm SP 19.0 Cat C with PG 58-28	150 mm Granular A (400 mm[1])	300 mm of Granular B, Type II
		Shattered Bedrock	40 mm SP 9.5 Cat C with PG 58-28	80 mm SP 19.0 Cat C with PG 58-28	150 mm Granular A (300 mm[1])	200 mm of Granular B, Type II
	Composite	Clayey Silt	50 mm SP 9.5 Cat C with PG 58-28	150 mm 30 MPa Concrete	200 mm Granular A	-
		Clay	50 mm SP 9.5 Cat C with PG 58-28	150 mm 30 MPa Concrete	150 mm Granular A	-
		Shattered Bedrock	50 mm SP 9.5 Cat C with PG 58-28	150 mm 30 MPa Concrete	150 mm Granular A	-
Collector (No Busses)	Flexible	Clayey Silt	40/50 mm SP 12.5 Cat C with PG 58-28	100/90 mm SP 19.0 Cat C with PG 58-28	150 mm Granular A (550 mm[1])	450 mm of Granular B, Type II
		Clay	40/50 mm SP 12.5 Cat C with PG 58-28	100/90 mm SP 19.0 Cat C with PG 58-28	150 mm Granular A (475 mm[1])	375 mm of Granular B, Type II
		Shattered Bedrock	40/50 mm SP 12.5 Cat C with PG 58-28	100/90 mm SP 19.0 Cat C with PG 58-28	150 mm Granular A (350 mm[1])	250 mm of Granular B, Type II
	Composite	Clayey Silt	50 mm SP 12.5 Cat C with PG 58-28	175 mm 30 MPa Concrete	200 mm Granular A	
		Clay	50 mm SP 12.5 Cat C with PG 58-28	175 mm 30 MPa Concrete	150 mm Granular A	
		Shattered Bedrock	50 mm SP 12.5 Cat C with PG 58-28	175 mm 30 MPa Concrete	150 mm Granular A	
Collector (Low Busses)	Flexible	Clayey Silt	40/50 mm SP 12.5 Cat C with PG 58-28	100/90 mm SP 19.0 Cat C with PG 58-28	150 mm Granular A (550 mm[1])	450 mm of Granular B, Type II
		Clay	40/50 mm SP 12.5 Cat C with PG 58-28	100/90 mm SP 19.0 Cat C with PG 58-28	150 mm Granular A (475 mm[1])	375 mm of Granular B, Type II
		Shattered Bedrock	40/50 mm SP 12.5 Cat C with PG 58-28	100/90 mm SP 19.0 Cat C with PG 58-28	150 mm Granular A (350 mm[1])	250 mm of Granular B, Type II
	Composite	Clayey Silt	50 mm SP 12.5 Cat C with PG 58-28	175 mm 30 MPa Concrete	200 mm Granular A	
		Clay	50 mm SP 12.5 Cat C with PG 58-28	175 mm 30 MPa Concrete	150 mm Granular A	
		Shattered Bedrock	50 mm SP 12.5 Cat C with PG 58-28	175 mm 30 MPa Concrete	150 mm Granular A	
Collector (High Busses)	Flexible	Clayey Silt	40/50 mm SP 12.5 Cat D with PG 64-28	100/90 mm SP 19.0 Cat D with PG 64-28	150 mm Granular A (550 mm[1])	450 mm of Granular B, Type II
		Clay	40/50 mm SP 12.5 Cat D with PG 64-28	100/90 mm SP 19.0 Cat D with PG 64-28	150 mm Granular A (475 mm[1])	375 mm of Granular B, Type II
		Shattered Bedrock	40/50 mm SP 12.5 Cat D with PG 64-28	100/90 mm SP 19.0 Cat D with PG 64-28	150 mm Granular A (350 mm[1])	250 mm of Granular B, Type II
	Composite	Clayey Silt	50 mm SP 12.5 Cat D with PG 64-28	150 mm 30 MPa Concrete	200 mm Granular A	-
		Clay	50 mm SP 12.5 Cat D with PG 64-28	150 mm 30 MPa Concrete	150 mm Granular A	-
		Shattered Bedrock	50 mm SP 12.5 Cat D with PG 64-28	150 mm 30 MPa Concrete	150 mm Granular A	-
Minor Arterial & Commercial/ Industrial	Flexible	Clayey Silt	40/50 mm SP 12.5 FC1 Cat D with PG 64-28	120/110 mm SP 19.0 Cat D with PG 64-28	150 mm Granular A (600 mm[1])	550 mm of Granular B, Type II
		Clay	40/50 mm SP 12.5 FC1 Cat D with PG 64-28	120/110 mm SP 19.0 Cat D with PG 64-28	150 mm Granular A (525 mm[1])	450 mm of Granular B, Type II
		Shattered Bedrock	40/50 mm SP 12.5 FC1 Cat D with PG 64-28	120/110 mm SP 19.0 Cat D with PG 64-28	150 mm Granular A (400 mm[1])	300 mm of Granular B, Type II
	Composite	Clayey Silt	50 mm SP 12.5 FC1 Cat D with PG 64-28	200 mm 30 MPa Concrete	250 mm Granular A	
		Clay	50 mm SP 12.5 FC1 Cat D with PG 64-28	200 mm 30 MPa Concrete	250 mm Granular A	
		Shattered Bedrock	50 mm SP 12.5 FC1 Cat D with PG 64-28	200 mm 30 MPa Concrete	150 mm Granular A	
Arterial	Flexible	Clayey Silt	40/50 mm SP 12.5 FC2 Cat D/E with PG 64-28	120/110 mm SP 19.0 Cat D/E with PG 64-28	150 mm Granular A (600 mm[1])	550 mm of Granular B, Type II
		Clay	40/50 mm SP 12.5 FC2 Cat D/E with PG 64-28	120/110 mm SP 19.0 Cat D/E with PG 64-28	150 mm Granular A (525 mm[1])	450 mm of Granular B, Type II
		Shattered Bedrock	40/50 mm SP 12.5 FC2 Cat D/E with PG 64-28	120/110 mm SP 19.0 Cat D/E with PG 64-28	150 mm Granular A (400 mm[1])	300 mm of Granular B, Type II
	Composite	Clayey Silt	50 mm SP 12.5 FC2 Cat D/E with PG 64-28	200 mm 30 MPa Concrete	250 mm Granular A	-
		Clay	50 mm SP 12.5 FC2 Cat D/E with PG 64-28	200 mm 30 MPa Concrete	250 mm Granular A	-
		Shattered Bedrock	50 mm SP 12.5 FC2 Cat D/E with PG 64-28	200 mm 30 MPa Concrete	150 mm Granular A	-
	Rigid	Clayey Silt	250 mm 30 MPa Concrete	-	300 mm Granular A	-
		Clay	250 mm 30 MPa Concrete	-	200 mm Granular A	-
		Shattered Bedrock	250 mm 30 MPa Concrete	-	200 mm Granular A	-

Note:
1. Total thickness of Granular A if no Granular B Type II is used.
2. All concrete layers should be non-reinforced.
3. Cells highlighted in green represent designs included in previous matrix

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APPENDIX D

AASHTO Pavement Design Sheets

1997 AASHTO Pavement Design

DARWin Pavement Design and Analysis System

A Proprietary AASHTOWare Computer Software Product

Golder Associates Ltd.

Flexible Structural Design Module

Local Roads
Flexible Pavement Design
Clay Subgrade

Flexible Structural Design

80-kN ESALs Over Initial Performance Period	2,000,000
Initial Serviceability	4.5
Terminal Serviceability	2.3
Reliability Level	85 %
Overall Standard Deviation	0.45
Roadbed Soil Resilient Modulus	30,000 kPa
Stage Construction	1
Calculated Design Structural Number	108 mm

Specified Layer Design

Layer	Material Description	Struct Coef. (Ai)	Drain Coef. (Mi)	Thickness (Di)(mm)	Width (m)	Calculated SN (mm)
1	Hot Mix Asphalt	0.44	1	120	-	53
2	Granular A	0.14	1	150	-	21
3	Granular B, Type II	0.12	1	300	-	36
Total	-	-	-	570	-	110

1997 AASHTO Pavement Design

DARWin Pavement Design and Analysis System

A Proprietary AASHTOWare Computer Software Product

Golder Associates Ltd.

Flexible Structural Design Module

Local Roads
Flexible Pavement Design
Clayey Silt Subgrade

Flexible Structural Design

80-kN ESALs Over Initial Performance Period	2,000,000
Initial Serviceability	4.5
Terminal Serviceability	2.3
Reliability Level	85 %
Overall Standard Deviation	0.45
Roadbed Soil Resilient Modulus	25,000 kPa
Stage Construction	1
Calculated Design Structural Number	114 mm

Specified Layer Design

Layer	Material Description	Struct Coef. (Ai)	Drain Coef. (Mi)	Thickness (Di)(mm)	Width (m)	Calculated SN (mm)
1	Hot Mix Asphalt	0.44	1	120	-	53
2	Granular A	0.14	1	150	-	21
3	Granular B, Type II	0.12	1	400	-	48
Total	-	-	-	670	-	122

1997 AASHTO Pavement Design

DARWin Pavement Design and Analysis System

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Computer Software Product

Golder Associates Ltd.

Flexible Structural Design Module

Local Roads
Flexible Pavement Design
Shallow Bedrock Subgrade

Flexible Structural Design

80-kN ESALs Over Initial Performance Period	2,000,000
Initial Serviceability	4.5
Terminal Serviceability	2.3
Reliability Level	85 %
Overall Standard Deviation	0.45
Roadbed Soil Resilient Modulus	70,000 kPa
Stage Construction	1
Calculated Design Structural Number	81 mm

Specified Layer Design

Layer	Material Description	Struct Coef. (Ai)	Drain Coef. (Mi)	Thickness (Di)(mm)	Width (m)	Calculated SN (mm)
1	Hot Mix Asphalt	0.44	1	120	-	53
2	Granular A	0.14	1	150	-	21
3	Granular B, Type II	0.12	1	200	-	24
Total	-	-	-	470	-	98

1997 AASHTO Pavement Design

DARWin Pavement Design and Analysis System

A Proprietary AASHTOWare Computer Software Product

Golder Associates Ltd.

Flexible Structural Design Module

Collector Roads
Flexible Pavement Design
Clay Subgrade

Flexible Structural Design

80-kN ESALs Over Initial Performance Period	4,500,000
Initial Serviceability	4.5
Terminal Serviceability	2.5
Reliability Level	90 %
Overall Standard Deviation	0.45
Roadbed Soil Resilient Modulus	30,000 kPa
Stage Construction	1
Calculated Design Structural Number	128 mm

Specified Layer Design

Layer	Material Description	Struct Coef. (Ai)	Drain Coef. (Mi)	Thickness (Di)(mm)	Width (m)	Calculated SN (mm)
1	Hot Mix Asphalt	0.44	1	140	-	62
2	Granular A	0.14	1	150	-	21
3	Granular B, Type II	0.12	1	375	-	45
Total	-	-	-	665	-	128

1997 AASHTO Pavement Design

DARWin Pavement Design and Analysis System

A Proprietary AASHTOWare Computer Software Product

Golder Associates Ltd.

Flexible Structural Design Module

Collector Roads
Flexible Pavement Design
Clayey Silt Subgrade

Flexible Structural Design

80-kN ESALs Over Initial Performance Period	4,500,000
Initial Serviceability	4.5
Terminal Serviceability	2.5
Reliability Level	90 %
Overall Standard Deviation	0.45
Roadbed Soil Resilient Modulus	25,000 kPa
Stage Construction	1
Calculated Design Structural Number	136 mm

Specified Layer Design

Layer	Material Description	Struct Coef. (Ai)	Drain Coef. (Mi)	Thickness (Di)(mm)	Width (m)	Calculated SN (mm)
1	Hot Mix Asphalt	0.44	1	140	-	62
2	Granular A	0.14	1	150	-	21
3	Granular B, Type II	0.12	1	450	-	54
Total	-	-	-	740	-	137

1997 AASHTO Pavement Design

DARWin Pavement Design and Analysis System

A Proprietary AASHTOWare Computer Software Product

Golder Associates Ltd.

Flexible Structural Design Module

Collector Roads
Flexible Pavement Design
Shallow Bedrock Subgrade

Flexible Structural Design

80-kN ESALs Over Initial Performance Period	4,500,000
Initial Serviceability	4.5
Terminal Serviceability	2.5
Reliability Level	90 %
Overall Standard Deviation	0.45
Roadbed Soil Resilient Modulus	70,000 kPa
Stage Construction	1
Calculated Design Structural Number	97 mm

Specified Layer Design

Layer	Material Description	Struct Coef. (Ai)	Drain Coef. (Mi)	Thickness (Di)(mm)	Width (m)	Calculated SN (mm)
1	Hot Mix Asphalt	0.44	1	140	-	62
2	Granular A	0.14	1	150	-	21
3	Granular B, Type II	0.12	1	250	-	30
Total	-	-	-	540	-	113

1997 AASHTO Pavement Design

DARWin Pavement Design and Analysis System

A Proprietary AASHTOWare Computer Software Product

Golder Associates Ltd.

Flexible Structural Design Module

Minor Arterial, Commercial/Industrial, and Arterial Roads
Flexible Pavement Design
Clay Subgrade

Flexible Structural Design

80-kN ESALs Over Initial Performance Period	11,000,000
Initial Serviceability	4.5
Terminal Serviceability	2.5
Reliability Level	90 %
Overall Standard Deviation	0.45
Roadbed Soil Resilient Modulus	30,000 kPa
Stage Construction	1
Calculated Design Structural Number	145 mm

Specified Layer Design

Layer	Material Description	Struct Coef. (Ai)	Drain Coef. (Mi)	Thickness (Di)(mm)	Width (m)	Calculated SN (mm)
1	Hot Mix Asphalt	0.44	1	160	-	70
2	Granular A	0.14	1	150	-	21
3	Granular B, Type II	0.12	1	450	-	54
Total	-	-	-	760	-	145

1997 AASHTO Pavement Design

DARWin Pavement Design and Analysis System

A Proprietary AASHTOWare
Computer Software Product

Golder Associates Ltd.

Flexible Structural Design Module

Minor Arterial, Commercial/Industrial and Arterial Road
Flexible Pavement Design
Clayey Silt Subgrade

Flexible Structural Design

80-kN ESALs Over Initial Performance Period	11,000,000
Initial Serviceability	4.5
Terminal Serviceability	2.5
Reliability Level	90 %
Overall Standard Deviation	0.45
Roadbed Soil Resilient Modulus	25,000 kPa
Stage Construction	1
Calculated Design Structural Number	153 mm

Specified Layer Design

Layer	Material Description	Struct Coef. (Ai)	Drain Coef. (Mi)	Thickness (Di)(mm)	Width (m)	Calculated SN (mm)
1	Hot Mix Asphalt	0.44	1	160	-	70
2	Granular A	0.14	1	150	-	21
3	Granular B, Type II	0.12	1	550	-	66
Total	-	-	-	860	-	157

1997 AASHTO Pavement Design

DARWin Pavement Design and Analysis System

A Proprietary AASHTOWare Computer Software Product

Golder Associates Ltd.

Flexible Structural Design Module

Minor Arterial, Commercial/Industrial and Arterial Road
Flexible Pavement Design
Shallow Bedrock Subgrade

Flexible Structural Design

80-kN ESALs Over Initial Performance Period	11,000,000
Initial Serviceability	4.5
Terminal Serviceability	2.5
Reliability Level	90 %
Overall Standard Deviation	0.45
Roadbed Soil Resilient Modulus	70,000 kPa
Stage Construction	1
Calculated Design Structural Number	110 mm

Specified Layer Design

Layer	Material Description	Struct Coef. (Ai)	Drain Coef. (Mi)	Thickness (Di)(mm)	Width (m)	Calculated SN (mm)
1	Hot Mix Asphalt	0.44	1	160	-	70
2	Granular A	0.14	1	150	-	21
3	Granular B, Type II	0.12	1	300	-	36
Total	-	-	-	610	-	127

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