Chapter 3: Point Source Technologies

3.1 Introduction

This chapter is a summary of the information gathered on the point source technologies and provides information on each technology that passed the screening process described in Chapter 2, *Technology/Indicator Analysis*. This information was used in the analysis of the screening alternatives and preliminary alternatives for both the Milwaukee Metropolitan Sewerage District (MMSD) 2020 Facilities Plan (2020 FP) and the Southeastern Wisconsin Regional Planning Commission (SEWRPC) Regional Water Quality Management Plan Update (RWQMPU), known collectively as the Water Quality Initiative (WQI).

In Chapter 2, *Technology/Indicator Analysis*, point source technologies were identified for a number of primary water quality indicators for further analysis using production theory. However, as the production function process was developed, it was determined that only the volume indicator produced production functions from which cost benefit curves could be developed. The other water quality indicators for which point source technologies were identified – total suspended solids (TSS), coliforms, nitrogen, and phosphorus - are limited because they are set at a fixed level of performance and do not produce a range of implementation. Therefore, the production function analysis focused on those point source technologies for which volume was the primary indicator.

The detailed analysis of the production function development for the technologies for which volume was the primary indicator is presented in Appendix 3A, *Point Source Technologies*. The summary of the analysis is presented below. The other point source technologies identified in Chapter 2, *Technology/Indicator Analysis* are also discussed in Appendix 3A. Costs are presented for these technologies, but they are not discussed in this chapter. Appendix 3A also identifies those technologies that were modified after they passed the screening process described in Chapter 2. Modifications (noted in Appendix 3A) made to improve the analysis included adjusting the original technology that was identified and combining multiple technologies into one set of production functions for which cost benefit curves were developed.

3.2 Volume Indicators

3.2.1 Analysis Development

The *State of the Art Report* (SOAR) analysis of the point source technologies is based on model simulations using the 64.5-year period of record (January 1940 through June 2004). The approach used in the SOAR analysis is to define a production function for each individual technology. The interactions between technologies are not studied in this analysis.

To define a production function, a series of simulations are run for each technology in which only one technology is varied at a time. The resulting production function characterizes the overall system response to the individual technology being analyzed. Each technology is applied to the fullest extent possible; in many cases the degree of implementation is far beyond the cost effective range of the technology. This is done to define production function curves in as wide a range as possible.

The results are used to compare and screen technologies to determine which ones merit further evaluation. The goal is to develop the unique cost benefit curves for each technology. The results of the SOAR analysis will be used to define screening alternatives and alternatives that will use combinations of technologies. Therefore, the SOAR analysis is a preliminary screening exercise that evaluates individual technologies; the alternatives will then be evaluated in more detail when they are grouped into combinations of technologies that have the greatest benefit.

The MACRO model was used to generate the production functions for this analysis. The MACRO model is a simple volumetric model of the MMSD system that accounts for treatment, storage in the inline storage system (ISS), and overflows. The model simulates the 64.5-year period of record and tests the sensitivity of the overall system response to various key parameters such as the treatment capacity, the volume of the ISS, pumping capacities, and operational conditions, such as the ISS volume reserved for separate sewer inflow (VRSSI).

3.2.2 Limitations of Analytical Methods

The MACRO model is a screening tool used to develop the production functions and test the relative sensitivity of the overall system response, such as combined sewer overflow (CSO) and sanitary sewer overflow (SSO) volume, to various technologies. MACRO is not a detailed design tool. It is simply used to generate production functions for a very wide range of conditions.

For detailed analysis of the system, the MOUSE model is available for simulating selected wet weather events. The MOUSE model is used to analyze the system response in the alternatives where greater detail is needed and the interaction between various technologies is important. Therefore, the MACRO results presented here should be understood as preliminary screening results of performance and benefit from the various technologies.

Ending SSOs or CSOs in this analysis only applies to the simulated conditions using the 64.5-year period of record. It is possible to experience conditions more extreme than those contained in the period of record. Extreme events are rare; therefore, a technology that will end overflows will rarely be used to the full extent. The cost to eliminate the largest event in the period of record is typically very large.

3.2.3 Summary Cost Benefit Curves

The primary results of the SOAR analysis of the point source technologies are presented as cost benefit curves. Full details on the development of the production functions for each technology and the cost functions used to create the cost benefit curves are presented in Appendix 3A, *Point Source Technologies* for each technology individually.

In the following figures, the cost benefit curves are presented for four point source indicators:

- SSO volume removed
- SSO events eliminated
- CSO volume removed

- CSO events eliminated

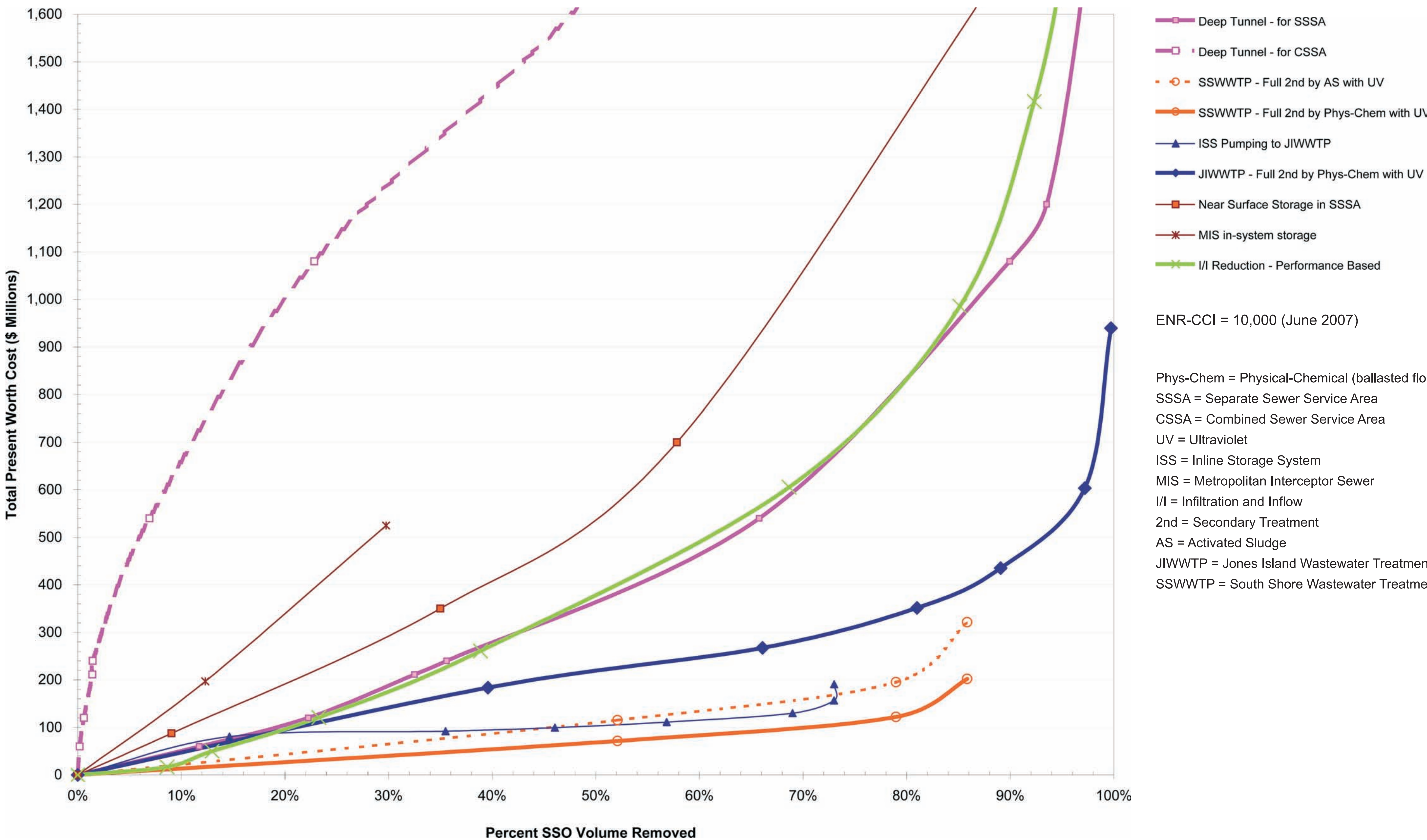
The total volume removed, which is the sum of CSO and SSO overflow volumes, is also shown as cost benefit curves.

Sanitary Sewer Overflow Volume Removed

The cost benefit curves for SSO volume removed are shown in Figure 3-1. This figure shows the most relevant technologies and compares the relative cost effectiveness of each technology and the maximum extent to which each technology can be applied. Technologies are assumed to be implemented until SSOs are ended even if the extent of implementation is highly unlikely. Some technologies cannot end SSOs due to other constraints. In those cases, the cost benefit curves extend as far as possible but do not completely end SSOs.

Sanitary Sewer Overflow Events Eliminated

The cost benefit curves for SSO events eliminated, as shown in Figure 3-2 compare the relative cost effectiveness of each technology.



ENR-CCI = 10,000 (June 2007)

Phys-Chem = Physical-Chemical (ballasted flocculation)
SSSA = Separate Sewer Service Area
CSSA = Combined Sewer Service Area
UV = Ultraviolet
ISS = Inline Storage System
MIS = Metropolitan Interceptor Sewer
I/I = Infiltration and Inflow
2nd = Secondary Treatment
AS = Activated Sludge
JIWWTP = Jones Island Wastewater Treatment Plant
SSWWTP = South Shore Wastewater Treatment Plant

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FIGURE 3-1
SSO VOLUME COST BENEFIT CURVES
2020 STATE OF THE ART REPORT
4/25/07 SOAR_03.0001.07.04.25.cdr

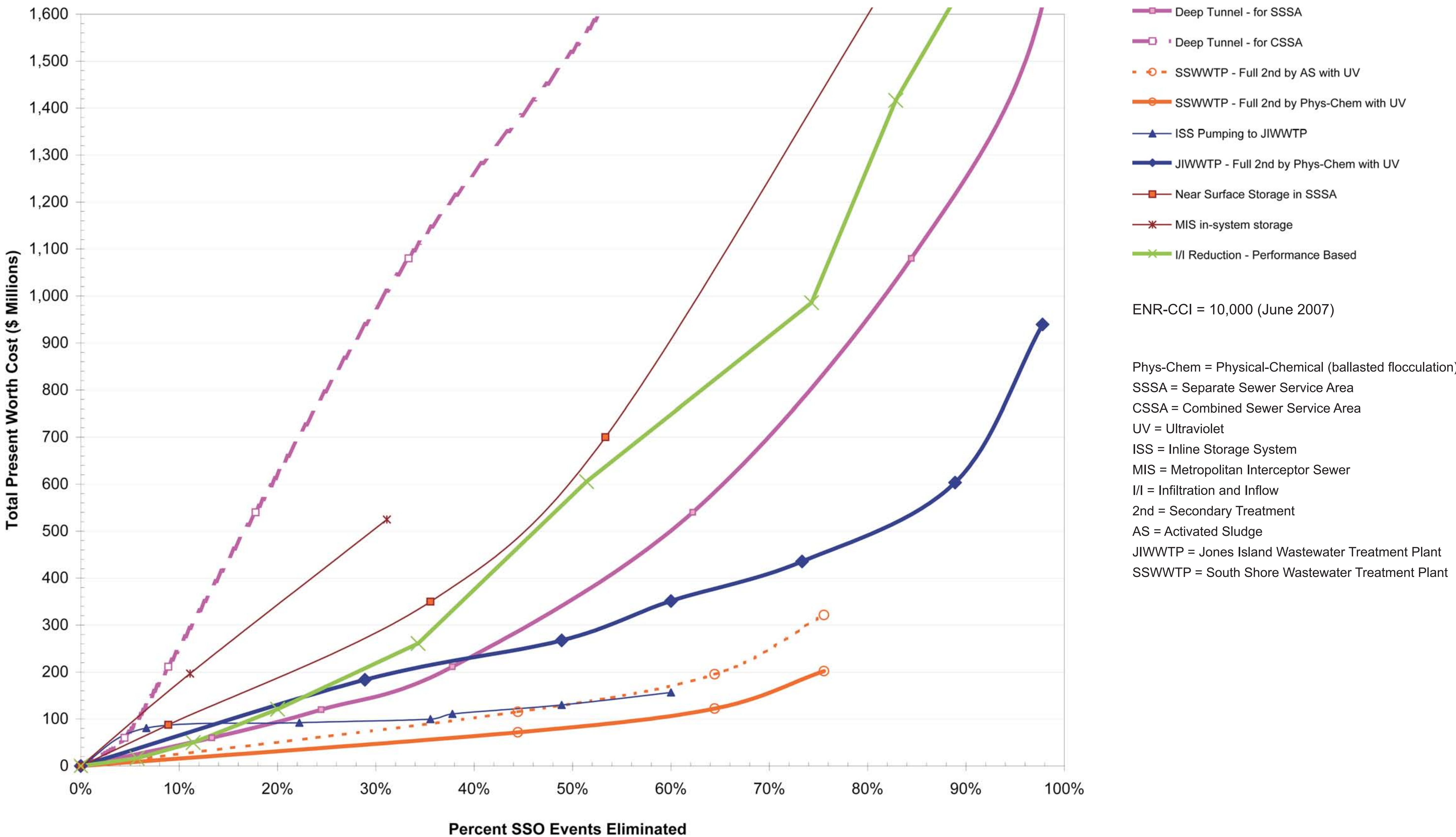

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FIGURE 3-2
SSO EVENTS COST BENEFIT CURVES
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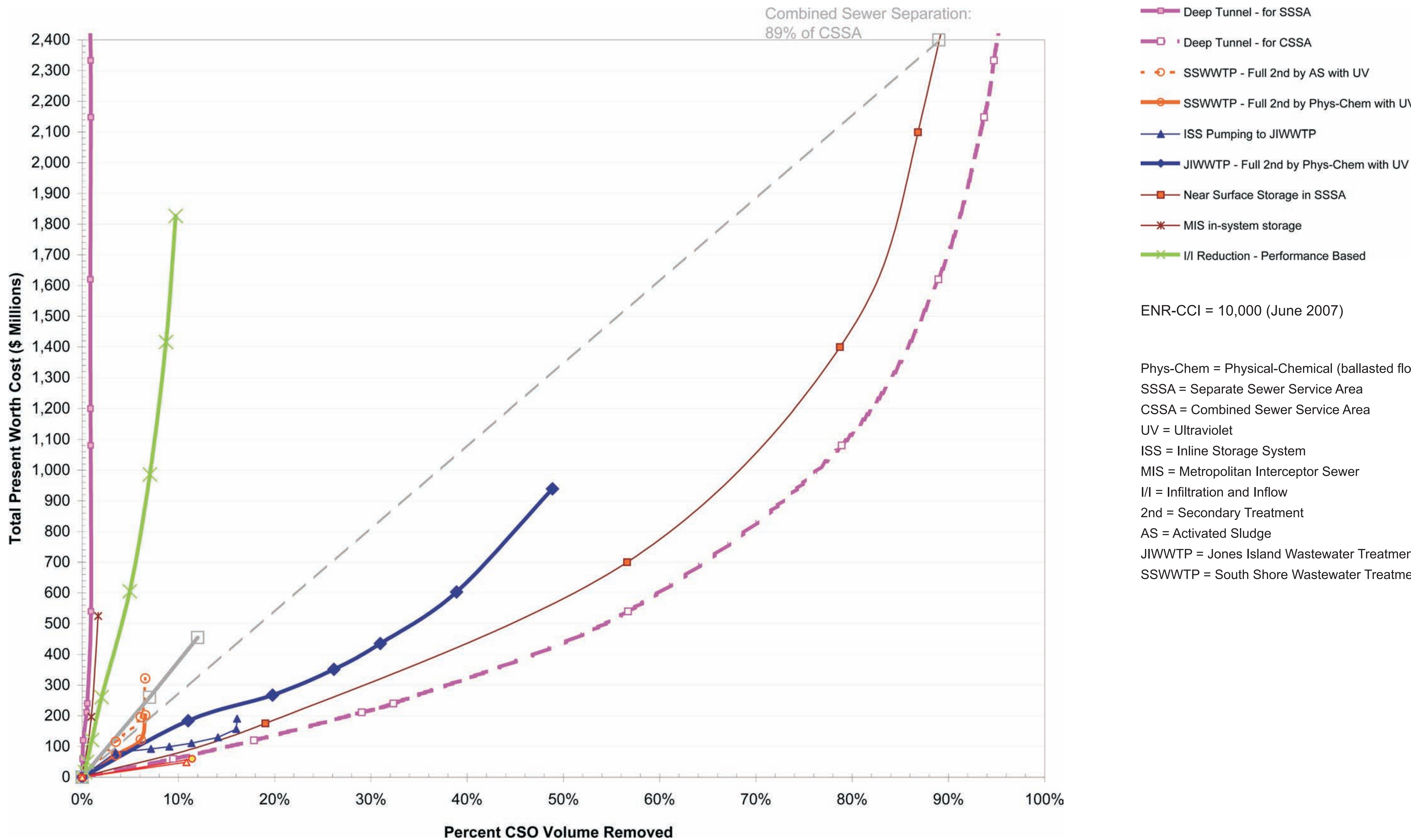
4/25/07 SOAR_03.0002.07.04.25.cdr

Combined Sewer Overflow Volume Removed

Figure 3-3 shows the cost benefit curves for CSO volume removed. Deep tunnel storage and near surface storage are the most cost effective technologies for CSO volume removed. An additional 2,200 MG of storage is needed to capture all CSO volume during the 64.5-year period of record. Figure 3-4 is a zoom-in view of Figure 3-3 for greater detail in the partial CSO volume removal range.

Combined Sewer Overflow Events Eliminated

Figure 3-5 shows the cost benefit curves for CSO events eliminated. The combined sewer separation technology removes CSO volume but it does not eliminate CSO events because 11% of the combined sewer area remains combined. Unless additional tunnel storage volume is added to contain the CSOs from the remaining area, the frequency of CSO events will remain approximately the same.



ENR-CCI = 10,000 (June 2007)

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FIGURE 3-3
CSO VOLUME COST BENEFIT CURVES
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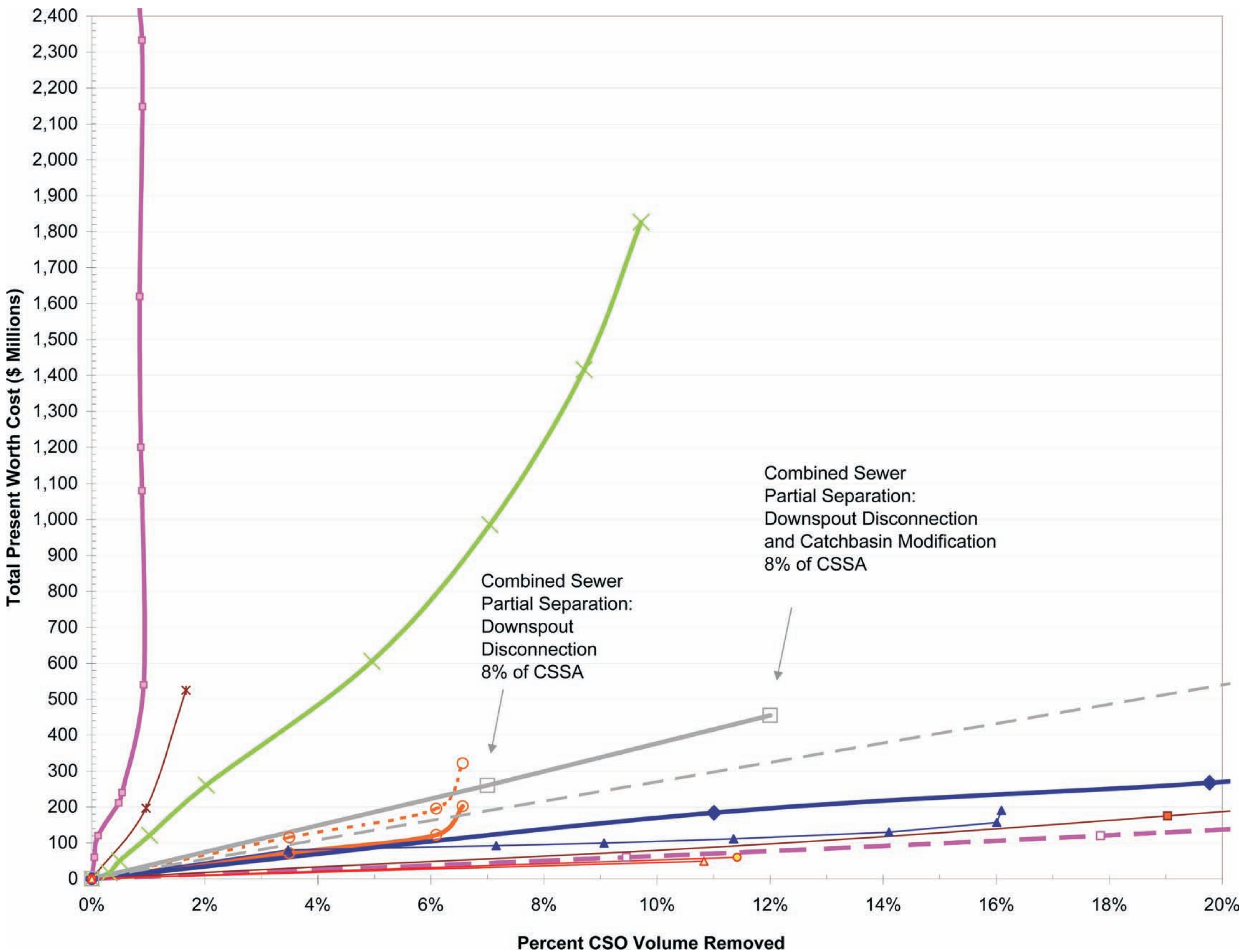

ENR-CCI = 10,000 (June 2007)

Phys-Chem = Physical-Chemical (ballasted flocculation)
SSSA = Separate Sewer Service Area
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FIGURE 3-4
(ZOOM-IN VIEW) CSO VOLUME COST BENEFIT CURVES
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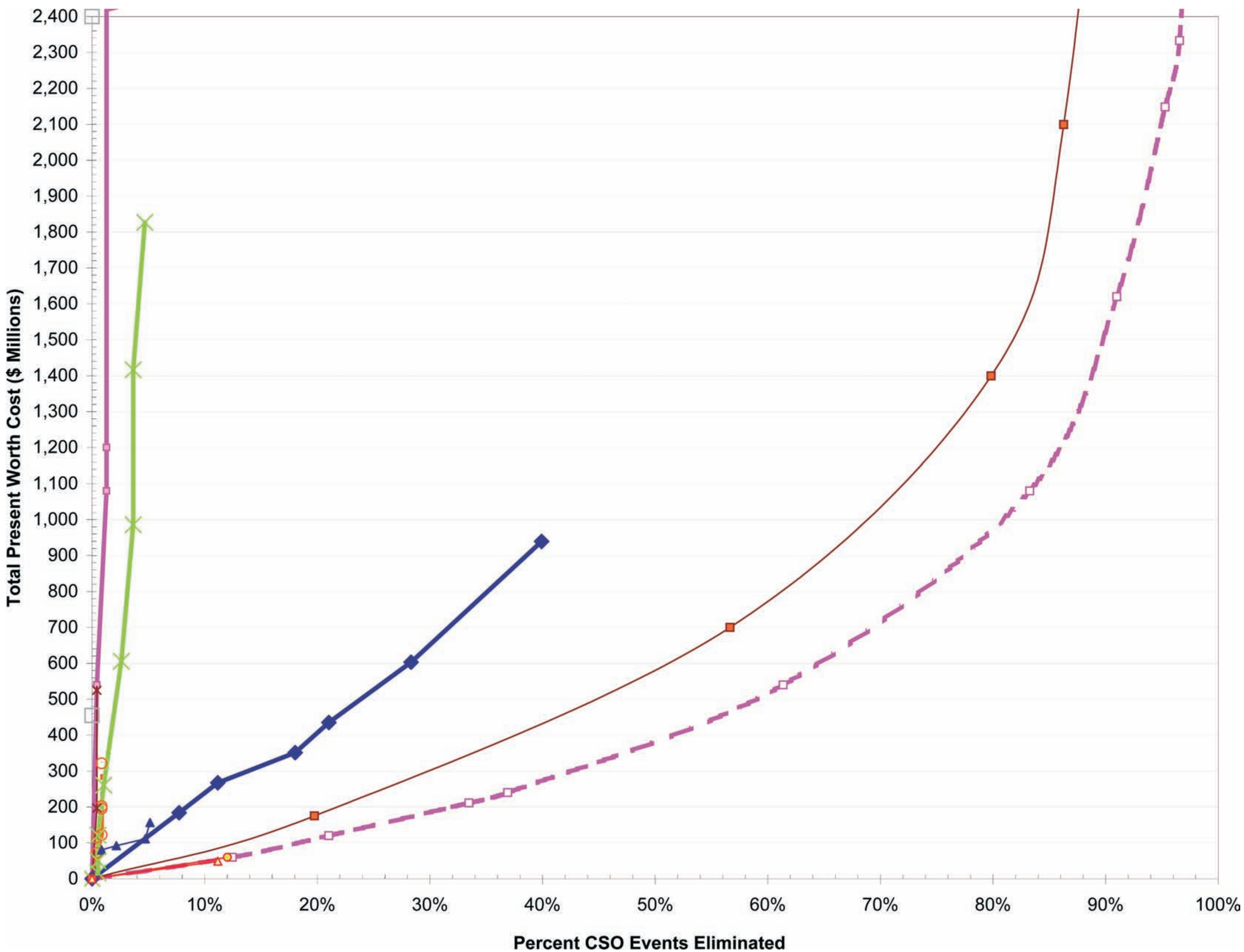
4/25/07 SOAR_03.0004.07.04.25.cdr



ENR-CCI = 10,000 (June 2007)

Phys-Chem = Physical-Chemical (ballasted flocculation)
SSSA = Separate Sewer Service Area
CSSA = Combined Sewer Service Area
UV = Ultraviolet
ISS = Inline Storage System
MIS = Metropolitan Interceptor Sewer
I/I = Infiltration and Inflow
2nd = Secondary Treatment
AS = Activated Sludge
JIWWTP = Jones Island Wastewater Treatment Plant
SSWWTP = South Shore Wastewater Treatment Plant

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FIGURE 3-5
CSO EVENTS COST BENEFIT CURVES
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4/25/07
SOAR_03.0005.07.04.25.cdr

The level of protection against SSOs or CSOs can be defined by the recurrence interval of the wet weather event. In the simulation results, the recurrence interval is determined by the number of CSO or SSO producing wet weather events during the simulation period. For example, a 10-year level of protection is equivalent to 6 to 7 events during the 64.5-year period of record. A 5-year level of protection is equivalent to about 13 events in 64.5 years.

Table 3-1 shows the relative approximate total present worth cost of the most cost effective technologies for various levels of protection. The table shows the costs for 5-, 10-, and 20-year levels of protection as well as a number of other levels of protection that are more relevant for CSO control. This analysis is based upon MACRO model results and shows relative differences between technologies to achieve the different results. For further analyses that refine these relative analyses, see the following sections of the *Facilities Plan Report*: Appendix 9A, *Screening Alternatives* and Chapter 9, *Alternative Analysis*.

Relative Total Present Worth Cost ($ Million)

SSO Events

Technology	Baseline (45 events)				5-yr LOP (13 events)	10-yr LOP (6 events)	20-yr LOP (3 events)	(1 event)	End SSOs (0 events in 64.5 years)
ISS pumping to JIWWTP					not achievable	not achievable	not achievable	not achievable	not achievable
SSWWTP, PCI, UV					$200	not achievable without forcemain pumping			
JIWWTP, PCI, UV with added pumping					$450	$600	$700	$1000	$1200
Deep Tunnel					$700	$1100	$1400	$1600	$2400
I/I Reduction					$2000	$2800	$4000	$4900	$6100

CSO Events

Technology	3.6 CSOs per yr (233 events)	3 CSOs per yr (194 events)	2 CSOs per yr (129 events)	1-yr LOP (65 events)	5-yr LOP (13 events)	10-yr LOP (6 events)	20-yr LOP (3 events)	(1 event)	End SSOs (0 events in 64.5 years)
ISS pumping to JIWWTP	0	not achievable							
SSWWTP, PCI, UV	0	not achievable							
JIWWTP, PCI, UV with added pumping	0	$300	not achievable						
Deep Tunnel	0	$80	$320	$780	$2,000	$3,100	$3,600	$4,000	$5,000
Near Surface Storage	0	$100	$430	$950	> $6,000	not achievable			
I/I Reduction	0	not achievable							

Based on VRSSI = 150 MG

Screening level costs based on MACRO modeling.

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TABLE 3-1
RELATIVE TOTAL PRESENT WORTH COST FOR LEVELS OF PROTECTION: SSO AND CSO REDUCTION
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Other Technologies

The benefits of the remaining technologies (inlet restrictors, rooftop storage, and MIS in-system storage) are all limited. The production function curves have been extended to the maximum possible level of implementation, but even then the curves stop far short of providing a significant benefit to reduce or eliminate overflows. In each case, the practical level of implementation is likely to be much less.

Appendix 3A:
Point Source Technologies

3A.1 Introduction

This appendix is an inventory of the information gathered on the point source technologies and provides information on each technology that passed the screening process described in Chapter 2, *Technology/Indicator Analysis*. This information was used in the analysis of the screening alternatives and preliminary alternatives for both the Milwaukee Metropolitan Sewerage District (MMSD) 2020 Facilities Plan (2020 FP) and the Southeastern Wisconsin Regional Planning Commission (SEWRPC) Regional Water Quality Management Plan Update (RWQMPU), known collectively as the Water Quality Initiative (WQI). Some technologies were added as the planning effort proceeded because new information was discovered or a particular need for the technology was identified. These technologies did not go through the screening process described in Chapter 2.

In Chapter 2, *Technology/Indicator Analysis*, point source technologies were identified for a number of primary water quality indicators for further analysis using production theory. However, as the production function process was developed, it was determined that only the volume water quality indicator produced production functions from which cost benefit curves could be developed. The other water quality indicators for which point source technologies were identified – total suspended solids (TSS), coliforms, nitrogen, and phosphorus - are limited because they are set at a fixed level of performance and do not produce a range of implementation. Therefore, the production function analysis focused on those point source technologies for which volume was the primary indicator.

3A.1.1 Volume Indicators

The volume indicator was broken into overflow volume and frequency of events (combined sewer overflows (CSO) and sanitary sewer overflows (SSO)) as the performance indicators. The benefits of each technology are evaluated for CSO and SSO overflow volume and the number of CSO and SSO overflow events. Separate sewer overflows can take place at a number of overflow sites and discharge into the receiving water bodies (creeks, rivers, and estuary). An SSO is counted as one event whether it occurs at one or more sites during a rainfall event; therefore, the number of SSO events does not have to do with the number of sites that experienced overflows, but with the presence of SSO discharge anywhere in the system. The same is true for the number of CSO events; it is not a site-specific count. The model does not distinguish between individual SSO sites; it only accounts for the total SSO volume and consequently the number of SSO events is based on the number of SSOs identified in the simulation results. Likewise, the model only accounts for the total CSO volume and is not site specific. The conveyance model evaluates the overall conveyance system response based on the volume of overflows.

In addition to the SSO and CSO volume indicators, results are presented for total overflow volume, which is the sum of the SSO and CSO volumes.

Although not included in the SOAR analysis, the response to the instream water quality, based on the reduction of overflows, is evaluated with the water quality model (Loading Simulation Program in C++, or LSPC). The SSO and CSO pollutant concentrations used in the water quality model are shown in Tables 3A-1, 3A-2, and 3A-3. The impacts to water quality as a result of reduced overflows are discussed in detail in Chapter 9, *Alternative Analysis* of the *Facilities Plan Report* and Chapter IX, *Alternative Plan Description and Evaluation*, of the Southeastern Wisconsin Regional Planning Commission's (SEWRPC's) Planning Report No. 50, *A Regional Water Quality Management Plan Update for the Greater Milwaukee Watersheds.*

All technologies are evaluated on the same basis using the same methods to calculate results. Each technology is described by a production function and a cost function. These two functions are then integrated into a cost benefit curve. Thus, for each technology (and each indicator) there is a series of three curves.

A schematic of the process is shown in Figure 3A-1. The cost benefit relationships are the essential products of this work because they use a common set of indicators to compare all of the point source technologies in a format that is independent of the specific variables of each technology.

3A.1.2 Other Technologies

The technologies that were identified for primary indicators other than volume are also discussed in this appendix. These indicators are limited by a set level of performance, so only one cost estimate was developed for each technology at the specific set level of performance depending on the performance indicator.

3A.1.3 Present Worth Development

All cost estimates determined the total present worth of the specific technology at the specific production or use level employed. For the technologies review for the volume indicator, multiple production or use levels could be developed that could be used to produce cost benefit relationships. For the other technologies, since only one production or use level could be developed, only one present worth could be developed. The present worth is calculated as follows:

- Total construction cost is determined at a facility plan accuracy level (+50% / -30%) as documented in the project file for each technology. Any references used are adjusted to an Engineering News Record Construction Cost Index (ENR-CCI) of 10,000, which is the estimate for Milwaukee as of June 2007.[a]
- A 25% contingency factor is uniformly applied to the construction cost to account for the possible missing elements inherent in a facility plan level cost estimate.
- A 30% cost factor is then applied to account for non construction costs such as design costs, construction management costs, permits, legal expenses and other administrative costs. The resulting cost is then categorized as the capital cost of the technology.
- An estimate is then made of operation and maintenance costs.

[a] This is the index published by ENR-CCI each month for various areas of the US – the value used in the 2020 FP is the average of the Chicago and Minneapolis values.

RECOMMENDED SSO MEAN CONCENTRATIONS FOR MODEL
(Converted Directly From MMSD and Wisconsin Community Sampling Data)

Parameter = Units =	BOD_5 (lb/MG)	Total Suspended Solids (lb/MG)	Fecal Coliform (#/MG)	E-coli (#/MG)	Total Phosphorus (lb/MG)	Copper (lb/MG)	Zinc (lb/MG)	Organic Nitrogen as N (lb/MG)
All Watersheds (Arithmetic Means)	426	1610	5.83E+13	3.6E+13	31	0.25	1.42	43
All Watersheds (Geometric Means)	217	793	1.7E+13	1.1E+13	21	0.17	1.08	28

RECOMMENDED CSO GEOMETRIC MEAN CONCENTRATIONS FOR MODEL
(Converted Directly From MMSD Sampling Data)

Parameter = Units =	BOD_5 (lb/MG)	Total Suspended Solids (lb/MG)	Fecal Coliform (#/MG)	E-coli (#/MG)	Total Phosphorus (lb/MG)	Copper (lb/MG)	Zinc (lb/MG)	Organic Nitrogen as N (lb/MG)	Ammonia as N (lb/MG)	Nitrate/ Nitrite as N (lb/MG)
Menomonee River (all but CT 5/6)	75	467	6.1E+12	3.6E+12	5.3	0.17	0.75	11	6	8
Menomonee River (only CT 5/6)	451	968	6.1E+12	3.6E+12	8.93	0.17	1.00	45	16	8
Kinnickinnic River	75	467	6.1E+12	3.6E+12	5.3	0.17	0.75	11	6	8
Milwaukee River	75	467	6.1E+12	3.6E+12	4.0	0.17	0.75	11	6	8

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TABLE 3A-2
SSO AND CSO POLLUTANT CONCENTRATIONS USED IN THE WATER QUALITY MODEL
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4/28/07 SOAR_3A.T002.07.04.28.cdr

RECOMMENDED CSO ARITHMETIC MEAN CONCENTRATIONS FOR MODEL
(Converted Directly From MMSD Sampling Data)

Parameter = Units =	BOD_5 (lb/MG)	Total Suspended Solids (lb/MG)	Fecal Coliform (#/MG)	E-coli (#/MG)	Total Phosphorus (lb/MG)	Copper (lb/MG)	Zinc (lb/MG)	Organic Nitrogen as N (lb/MG)
Menomonee River (all but CT 5/6)	117	734	2.5E+13	4.9E+12	6.93	0.17	0.83	14
Menomonee River (only CT 5/6)	1118	1435	2.5E+13	4.9E+12	12.2	0.17	1.42	107
Kinnickinnic River	117	734	2.5E+13	4.9E+12	6.68	0.17	0.83	14
Milwaukee River	117	734	2.5E+13	4.9E+12	4.84	0.17	0.83	14

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TABLE 3A-3
SSO AND CSO POLLUTANT CONCENTRATIONS USED IN THE WATER QUALITY MODEL
2020 STATE OF THE ART REPORT
4/28/07 SOAR_3A.T003.07.04.28.cdr

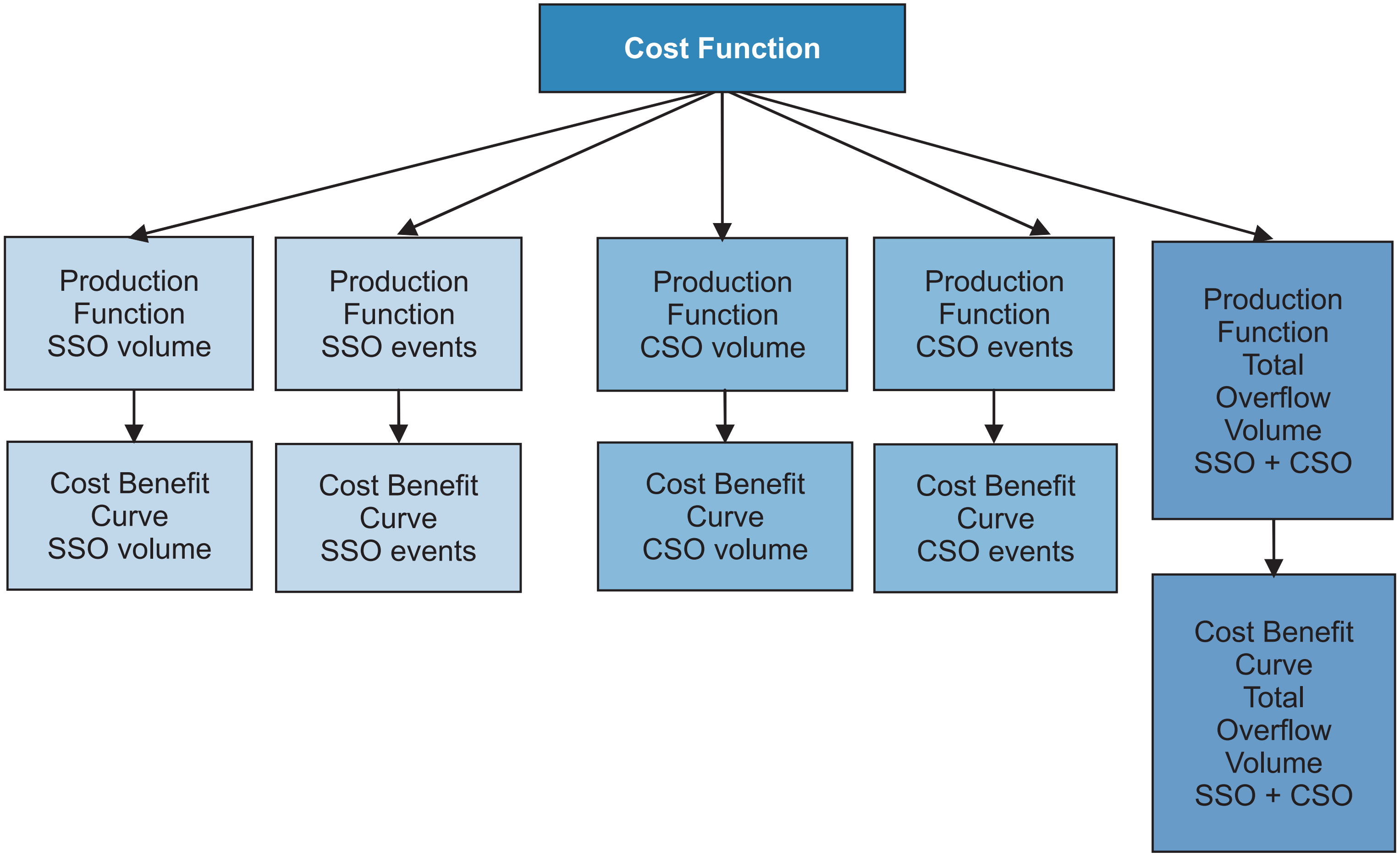

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FIGURE 3A-1
SCHEMATIC OF PROCESS TO CREATE THE COST BENEFIT CURVES
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4/26/07 SOAR_3A.0001.07.04.26.cdr

- Total present worth of the technology is then the sum of the capital cost and the 20 year present worth of the operating costs calculated at a discount rate of 5 1/8% - which is the current Wisconsin Department of Natural Resources (WDNR) Facilities Plan mandated discount rate. This calculation results in a total present worth of the capital costs plus 12.3 times the operating costs. The number 12.3 is the 20 year present worth factor for annual costs at 5 1/8%.

3A.1.4 Volume Indicator Production Functions Model Development

Model Configuration: 2020 Baseline Conditions and Flows

The volumes of CSO and SSO removed are relative to the 2020 Baseline case with the existing inline storage system (ISS) volume of 405 million gallons (MG) and the volume reserved for separate sewer inflow (VRSSI) assumed to be equal to 150 MG. The maximum tunnel pump out rate is assumed to be 80 million gallons per day (MGD) to Jones Island Wastewater Treatment Plant (JIWWTP) and 40 MGD to South Shore Wastewater Treatment Plant (SSWWTP). The capacity of SSWWTP is 300 MGD.

The capacity of JIWWTP is 330 MGD for the full treatment process (primary and secondary treatment). The blending limit is 60 MGD and the chlorination limit is 390 MGD. Therefore, the total capacity at JIWWTP is 390 MGD under peak conditions as stated in the current discharge permit.

The harbor siphon limit values in the model are based on the screw pump capacity of 140 MGD for the low level siphon and the "committed" hydraulic capacity of 180 MGD for the high level siphon. Therefore, the gravity flow to JIWWTP is limited to 320 MGD, even though the firm capacity of the high level screw pumps is 330 MGD.

The Northwest Side (NWS) Relief Sewer is included in the model as an 88 MG storage facility in the separate sewer service area (SSSA). The Port Washington Road Relief Sewer and the West Wisconsin Avenue Relief Sewer were not accounted for in the model because at the time the simulations were performed these projects were still in the early stages of definition. These facilities will provide additional capacity when constructed; therefore the modeled results are conservative.

The MACRO simulations used the hydrologic conditions at General Mitchell International Airport (GMIA) over the 64.5-year period of record. The flows are based on the estimated 2020 population and land use values prepared by SEWRPC. Table 3A-4 is a summary of the simulation conditions used for the baseline case.

TABLE 3A-4
BASELINE OPERATIONAL PARAMETERS AND SIMULATION CONDITIONS

Parameter	Value	
ISS Volume	405 MG	
VRSSI	150 MG	
SSWWTP Peak Capacity	300 MGD	
JIWWTP Peak Capacity	390 MGD	(includes up to 60 MGD of blending)
High Level Harbor Siphon Capacity	180 MGD	(committed siphon capacity)
Low Level Harbor Siphon Capacity	140 MGD	(low level screw pump limit)
ISS Pump Out Limit to JIWWTP	80 MGD	(two pumps)
ISS Pump Out Limit to SSWWTP	40 MGD	(one pump)
NWS Relief Sewer (Passive fill operations)	88 MG	(modeled as remote storage in the SSSA)
Population and Land Use Conditions	SEWRPC estimates for 2020 Baseline	
Meteorological Input Source	General Mitchell International Airport	

ISS = Inline Storage System
MG = Million Gallons
NWS = Northwest Side
SSSA = Separate Sewer Service Area
VRSSI = Volume Reserved for Separate Sewer Inflow
JIWWTP = Jones Island Wastewater Treatment Plant
MGD = Million Gallons per Day
SEWRPC = Southeastern Wisconsin Regional Planning Commission
SSWWTP = South Shore Wastewater Treatment Plant

Model Results: Baseline Case

For the baseline conditions, the total projected CSO volume is 60,800 MG (sum of 233 events) and the total projected SSO volume is 9,700 MG (sum of 45 tunnel-related events - overflow events that occur due to a lack of tunnel storage rather than other factors such as metropolitan interceptor sewer (MIS) capacity constraints). These volumes represent the accumulated CSO and SSO volumes generated in the MACRO simulation model using the hydrologic conditions at GMIA over the 64.5 year period of record.

On an annual basis, the average simulated CSO volume is approximately 940 MG and the average SSO volume is approximately 150 MG. The average simulated frequency of CSO events is 3.6 CSOs per year, and the average frequency of SSO events is 0.7 SSOs per year. The SSO frequency can also be expressed as the recurrence interval of simulated tunnel-related SSO events. The recurrence interval is the inverse of the number of events per year. The simulated SSO recurrence interval is 1.4 years for tunnel-related SSO events.

Model Configuration for Production Functions: VRSSI Assumptions

During wet weather, the combined sewer service area (CSSA) responds rapidly to rainfall because the response is dominated by surface runoff. The separate sewer service area (SSSA) has a slower response because infiltration is a significant component of the infiltration and inflow (I/I) in the SSSA. Consequently, the tunnel fills with inflow from the CSSA first. In the baseline case, the gates to the tunnel are closed to the combined sewer flow when the tunnel has 150 MG of remaining storage, i.e., the VRSSI is 150 MG.

For simulations with additional tunnel volume, the model was configured to generate production function curves for two cases depending on how the volume reserved for separate sewer inflow is managed. In the first case, the VRSSI is increased an amount equal to the additional tunnel volume. This case makes the additional tunnel volume available primarily for SSO reduction.

In the second case, the VRSSI is not increased, but remains a constant value of 150 MG. Consequently, the additional tunnel volume is used primarily for CSO reduction.

Additional Special Case: Initial VRSSI Equal to 200 MG

A special set of simulations were run using the initial VRSSI equal to 200 MG. These were done because MMSD revised the initial VRSSI to 200 MGD after the 2020 FP had already done a large amount of technology evaluations at 150 MGD, which was the VRSSI that MMSD used when the 2020 FP was initiated. As the tunnel volume was increased for this special set of simulations, the VRSSI was also increased an equivalent amount. Because more storage volume is reserved for separate sewer inflow, the SSO volume decreases and the CSO volume increases. The baseline case using 200 MG for VRSSI produced 75,500 MG of CSO (in 323 events) and 7,500 MG of SSO (in 34 events).

Along with additional tunnel volume in this special case, an equal amount of volume was added to the initial VRSSI. Because this special case has a new baseline condition starting with a VRSSI of 200 MG, the results for this case approach different limit values.

Results for all of the other technologies are presented relative to the baseline that used VRSSI equal to 150 MG. Therefore, the results of the special case with VRSSI equal to 200 MG can not be compared directly with those other technologies.

3A.2 Storage and Conveyance Technologies

3A.2.1 Technology: Deep Tunnel Storage

Introduction

The existing deep tunnel, called the inline storage system (ISS), has a volume of approximately 405 MG. It was completed in 1994 and has been very effective in reducing the volume and number of overflow events. This investigation evaluates the benefits of increasing the size of the existing tunnel.

Design Factors

In this analysis, the factors used to describe the deep tunnel technology are the additional tunnel volume and the VRSSI; both the tunnel volume and the VRSSI are varied. In Section 3A.2.3, *Technology: Volume Reserved for Separate Sewer Inflow*, VRSSI is studied as a stand alone technology without increasing the ISS volume.

The tunnel receives flow from both the CSSA and SSSA. The flows in the CSSA have higher peak flow rates and rapid response times to the rainfall when compared with the flows in the SSSA. Consequently, the tunnel fills with combined sewer flow first. To ensure that the necessary storage volume is available for separate sewer flow, the tunnel gates serving the CSSA close to preserve the necessary separate sewer storage volume. The volume reserved for separate sewer inflow to the tunnel is called VRSSI. In the model, the VRSSI is a fixed value; in reality, operators change the volume reserved using their experience and available data to optimize the use of the tunnel.

Tunnel volume and VRSSI are the only two tunnel characteristics used in the model to generate the production functions. Other design factors such as tunnel diameter, depth, length, and geological conditions are a part of the cost function development but are not explicit parameters used in modeling. This analysis is general and does not assume specific details such as the number or location of tunnels or the number of dropshafts.

Experience

Experience with the ISS demonstrates the important benefits of deep tunnel storage. The NWS Relief Sewer further adds to the Milwaukee Metropolitan Sewerage District (MMSD) experience using deep tunnel technology. This is a proven and effective technology.

Production Functions

The production functions for additional deep tunnel storage to reduce overflows were generated using the MACRO model of the MMSD system. MACRO is a simple, volumetric model used to simulate the overall response of the MMSD conveyance and treatment system. MACRO represents the overall system as two separate sewer units and two combined sewer units. The model is used for long-term simulations. The results were used to compute the CSO and SSO volumes removed and events eliminated during the 64.5 year period of record (January 1940 to June 2004).

One set of production functions represents the volume of CSO and SSO removed by additional deep tunnel volume. Another set of production functions represents the number of CSO and SSO events eliminated.

This section describes important assumptions used in the development of the production functions and limitations on their application.

Results: Sanitary Sewer Overflow and Combined Sewer Overflow Production Functions

Case 1 – Increasing Tunnel Volume and Volume Reserved for Separate Sewer Inflow

In Case 1, the additional VRSSI volume is equal to additional tunnel volume. Consequently, the additional tunnel volume is available for separate sewer flow. If the ISS volume is increased by an additional 450 MG, 90% of the SSO volume is eliminated. When the additional tunnel volume is 1,008 MG, the need for SSO storage is completely satisfied. The maximum value for VRSSI is 1,158 MG (the existing VRSSI of 150 MG plus the additional volume of 1,008 MG).

Further increases in tunnel volume are available for CSO reduction (for these conditions, the VRSSI does not increase beyond 1,158 MG, which is necessary to end SSOs). With 1,600 MG of additional volume (total ISS volume of 2,005 MG), 90% of the CSO volume is eliminated. All CSOs end with 3,148 MG of additional volume (total ISS volume of 3,553 MG).

To end both CSOs and SSOs, it is better to use the conditions of Case 2 rather than Case 1.

Case 2 – Increasing Tunnel Volume (with constant VRSSI equal to 150 MG)

When the VRSSI remains constant, the additional tunnel volume is available for both combined and separate sewer flows. Because the combined area flow responds more quickly, most of the additional tunnel volume is filled by the combined sewer flow. Consequently, the SSO reduction is less rapid compared with Case 1; 1,600 MG of additional tunnel volume is required to eliminate 90% of the SSO volume. The SSOs are ended with an additional 2,175 MG of storage (total ISS volume of 2,580 MG).

When the VRSSI remains constant, the CSO volume removed increases rapidly; 90% of the total CSO volume is eliminated with an additional 700 MG of tunnel volume. After this rapid gain, the curve flattens out so that 1,600 MG of additional storage is required to eliminate 99% of the CSO volume and 2,175 MG (total ISS volume of 2,580 MG) are required to end all CSOs and SSOs.

The results of these two cases show that if the goal is to end both CSOs and SSOs, the Case 2 approach can achieve the goal with a smaller tunnel volume.

Figure 3A-2 shows the production function curves for deep tunnel storage. Case 1 uses additional VRSSI; there are two curves for Case 1, one starting with VRSSI = 150 MG and the other for VRSSI = 200 MG. The Case 1 curves rise rapidly, which indicates that small additions to the tunnel produce substantial reductions in the SSO volume. Case 2 is less efficient at removing SSO because the additional tunnel volume is used to store flow from both the CSSA and the SSSA.

Figure 3A-3 presents the same information as Figure 3A-2, but in this figure the production function is a percent of the total SSO volume removed. In this format, the two curves with Case 1 conditions are coincident. To end SSOs, approximately 1,000 MG of additional tunnel storage is needed; it is not dependent on whether the initial VRSSI starts at 150 or 200 MG.

Figure 3A-4 shows the production functions for the CSO volume removed by additional tunnel storage. Case 2 is very efficient in CSO removal as shown by the rapid rise of the production function curve. The Case 1 curves show that there is no benefit for CSO reduction until all of the SSOs are eliminated. Then the remaining tunnel volume is available for CSO control.

Figure 3A-5 shows the production functions for the total overflow volume (both CSO and SSO) removed by additional tunnel storage. The Case 1 curves have a distinct transition point where the curves flatten out until all SSO volume is removed. Beyond the transition point, the additional tunnel volume is used for CSO removal until all overflows are eliminated. If one does not distinguish between CSOs and SSOs, but considers the total overflow volume as the sum of the CSO and SSO volume, then it is more efficient to reduce the total overflow volume by using a fixed VRSSI value (as in Case 2).

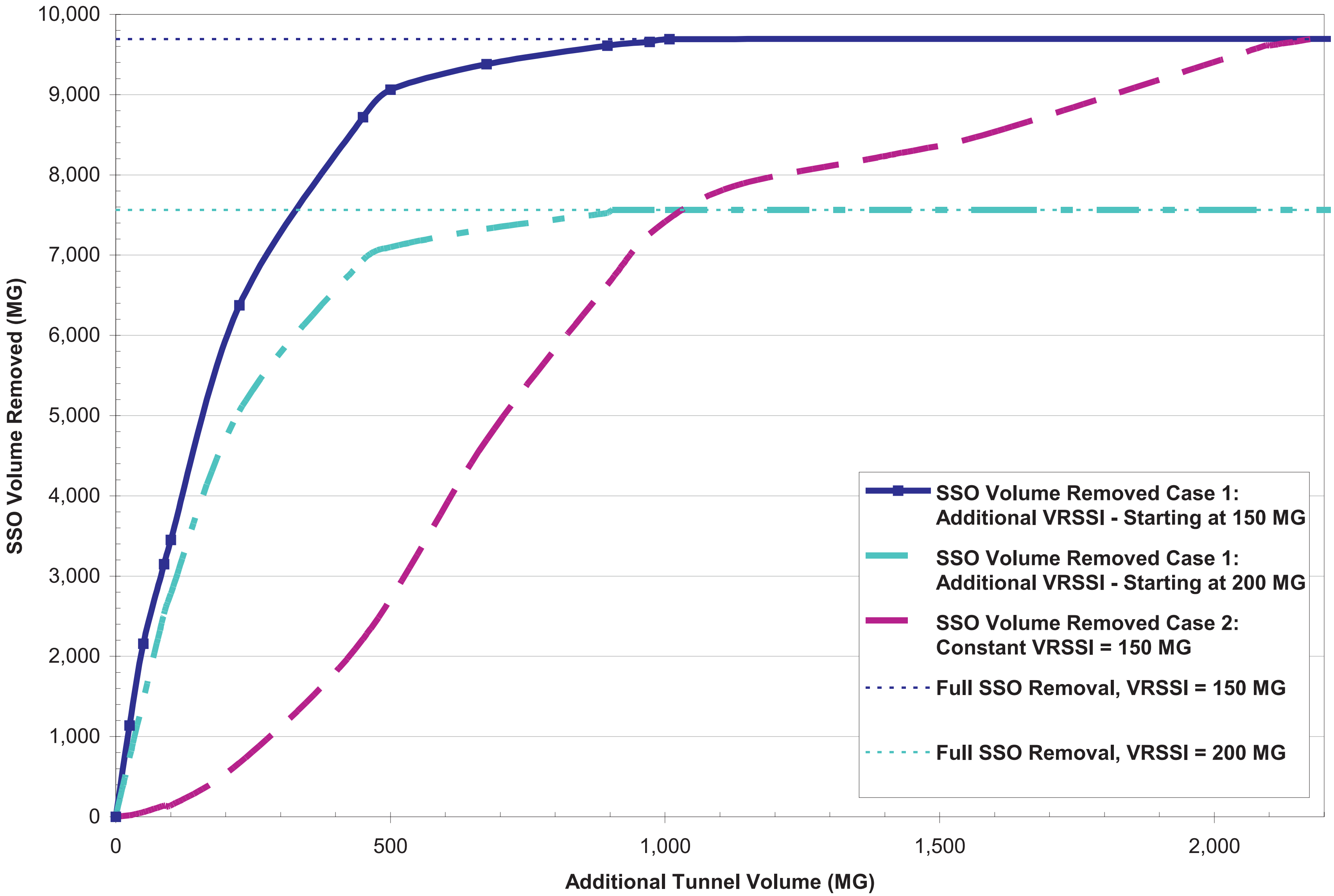

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FIGURE 3A-2
SSO VOLUME PRODUCTION FUNCTIONS: DEEP TUNNEL STORAGE
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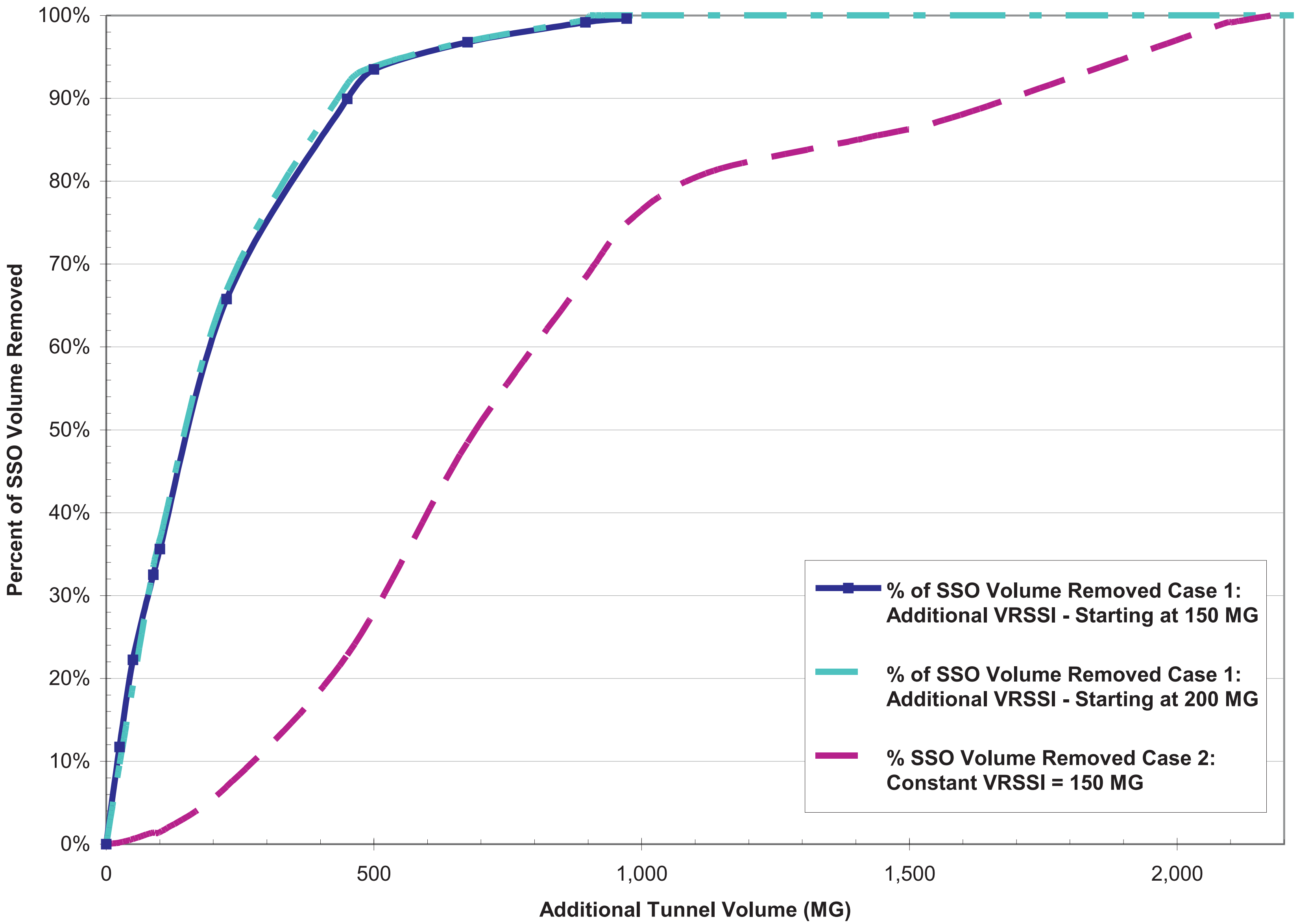

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FIGURE 3A-3
PERCENT SSO VOLUME PRODUCTION FUNCTIONS: DEEP TUNNEL STORAGE
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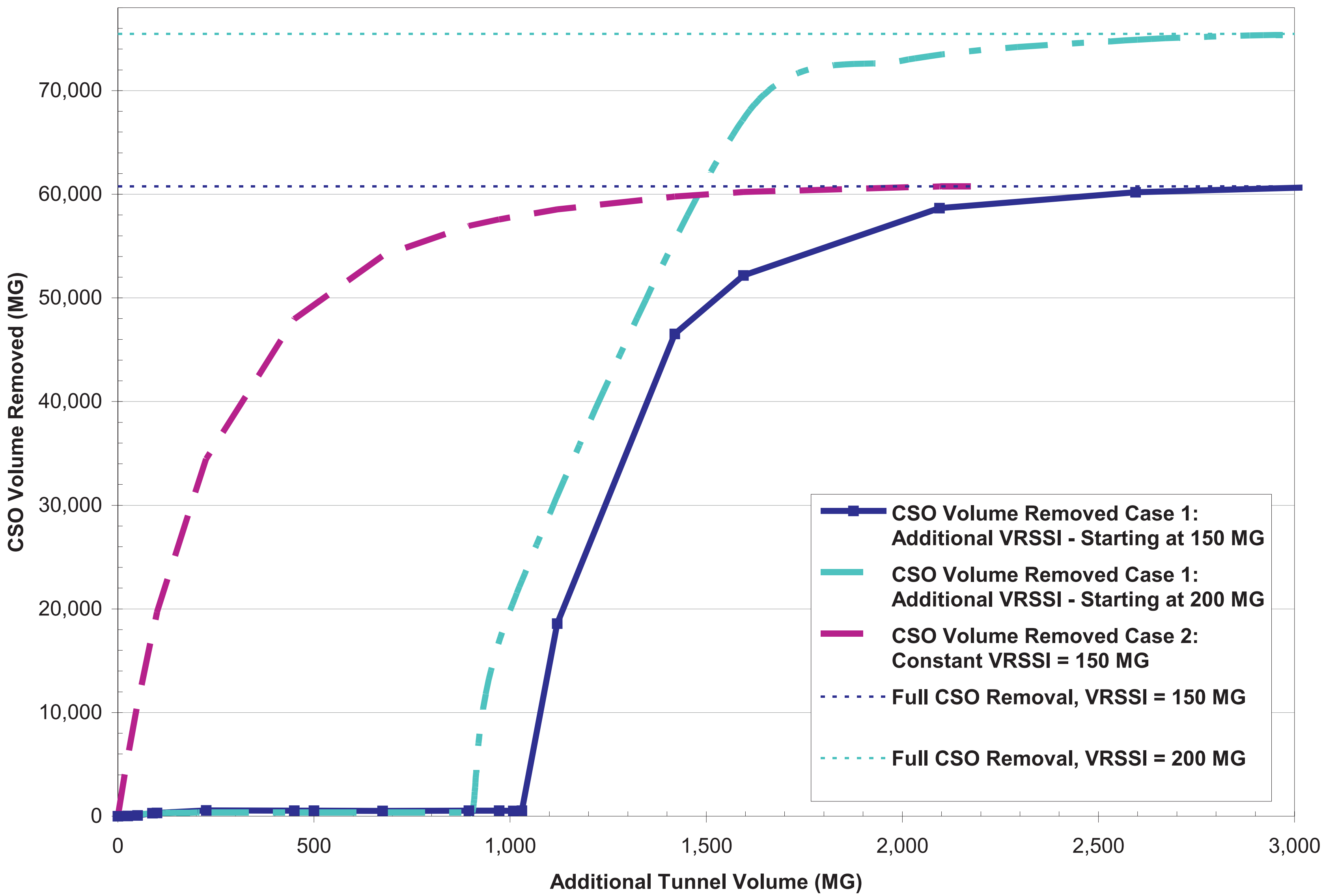

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FIGURE 3A-4

CSO VOLUME PRODUCTION FUNCTIONS: DEEP TUNNEL STORAGE

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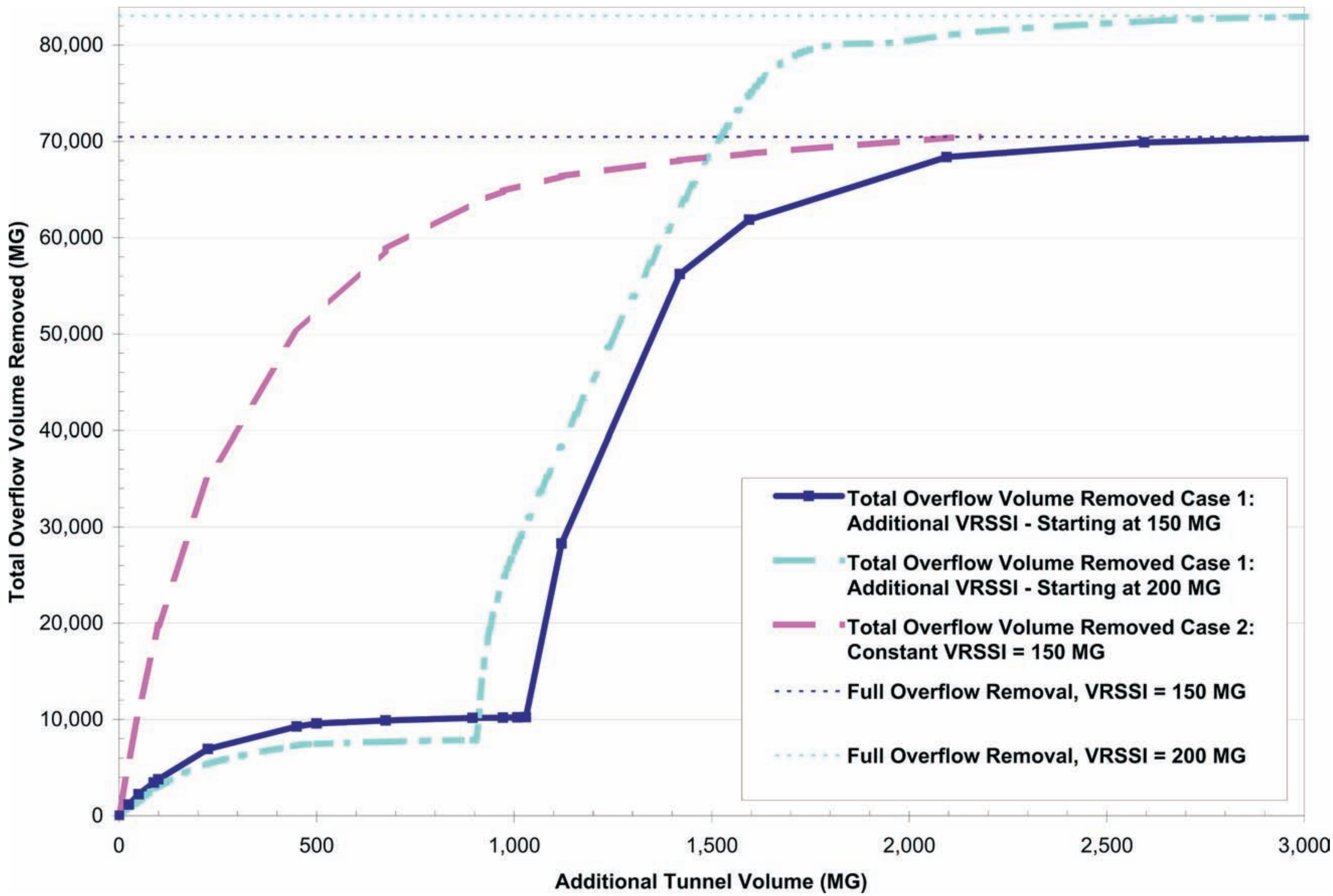

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FIGURE 3A-5

TOTAL OVERFLOW VOLUME PRODUCTION FUNCTIONS: DEEP TUNNEL STORAGE

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Figures 3A-6 and 3A-7 show the production functions for the number of SSO and CSO events eliminated by additional tunnel storage. The Case 1 curves show that SSO events are eliminated efficiently when the VRSSI increases with the additional tunnel volume. The Case 2 curves show that CSO events are eliminated efficiently when the VRSSI remains constant.

These cases are not the only possible combinations of tunnel size and VRSSI. These cases frame the problem but do not necessarily represent an optimum combination. It is likely that a case with approximately 500 MG of additional tunnel volume and only 250 MG of additional VRSSI may effectively reduce both CSOs and SSOs significantly, but will not end either completely. This type of question is addressed in Chapter 9, *Alternatives Analysis* of the *Facilities Plan Report*.

Results Interpretation

1) Eliminating SSOs and CSOs in the simulation applies to the 64.5-year period of record. For a storm larger than those experienced in the record, additional tunnel volume would be required to eliminate all overflows. It is not possible to define a tunnel volume that will prevent SSOs and CSOs for all hydrologic conditions. Because the curves rise rapidly at first and then flatten out, it is very costly to add sufficient volume to eliminate 100% of the overflows. Elimination of the first 90% of the volume is much more efficient than elimination of the final 10% of the volume.
2) Most SSOs in the MMSD system are related to tunnel closure (a few are related to site-specific MIS capacity limitations). SSOs due to site specific MIS limitations are not a significant fraction of the total SSO volume, but the absence of these SSOs can influence the number of SSO events reported. It is important to recognize the fact that MACRO results only represent the tunnel related SSOs. It is also important to recognize that additional tunnel volume is not effective in reducing SSOs due to a site specific MIS capacity limitation.

Cost Function

The cost function for additional deep tunnel volume represents the total costs of applying the technology. It includes the present worth of construction, technical services, operations and maintenance, and replacement costs for a 20-year service period. The costs are scaled to ENR-CCI = 10,000 (estimated to be 2007) with consistent assumptions for contingencies, technical services, and discount rates.

The cost function shown in Figure 3A-8 uses a constant unit cost of $2.40/gallon, which is appropriate for tunnels over 100 MG. The unit cost is higher for smaller tunnels; this is discussed below.

The cost function uses the following assumptions:

- Contingencies equal to 25% of basic construction cost
- Technical services for engineering and administration = 25% of total construction cost (including contingencies)
- 20 year operations and maintenance (O&M), including replacement costs
- O&M annual inflation rate: 5.375%
- Annual discount rate: 5.375%

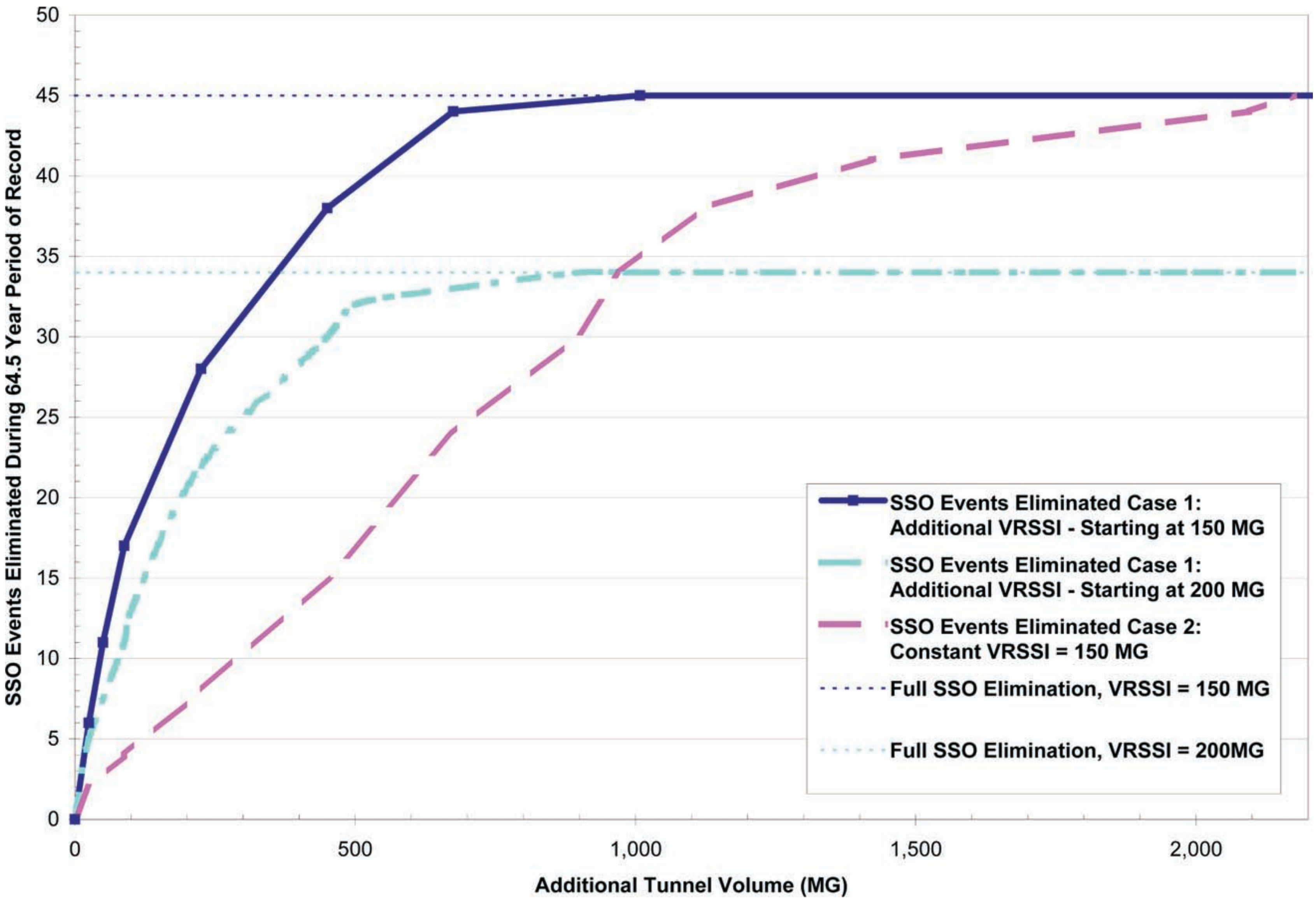

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FIGURE 3A-6

SSO EVENT PRODUCTION FUNCTIONS: DEEP TUNNEL STORAGE

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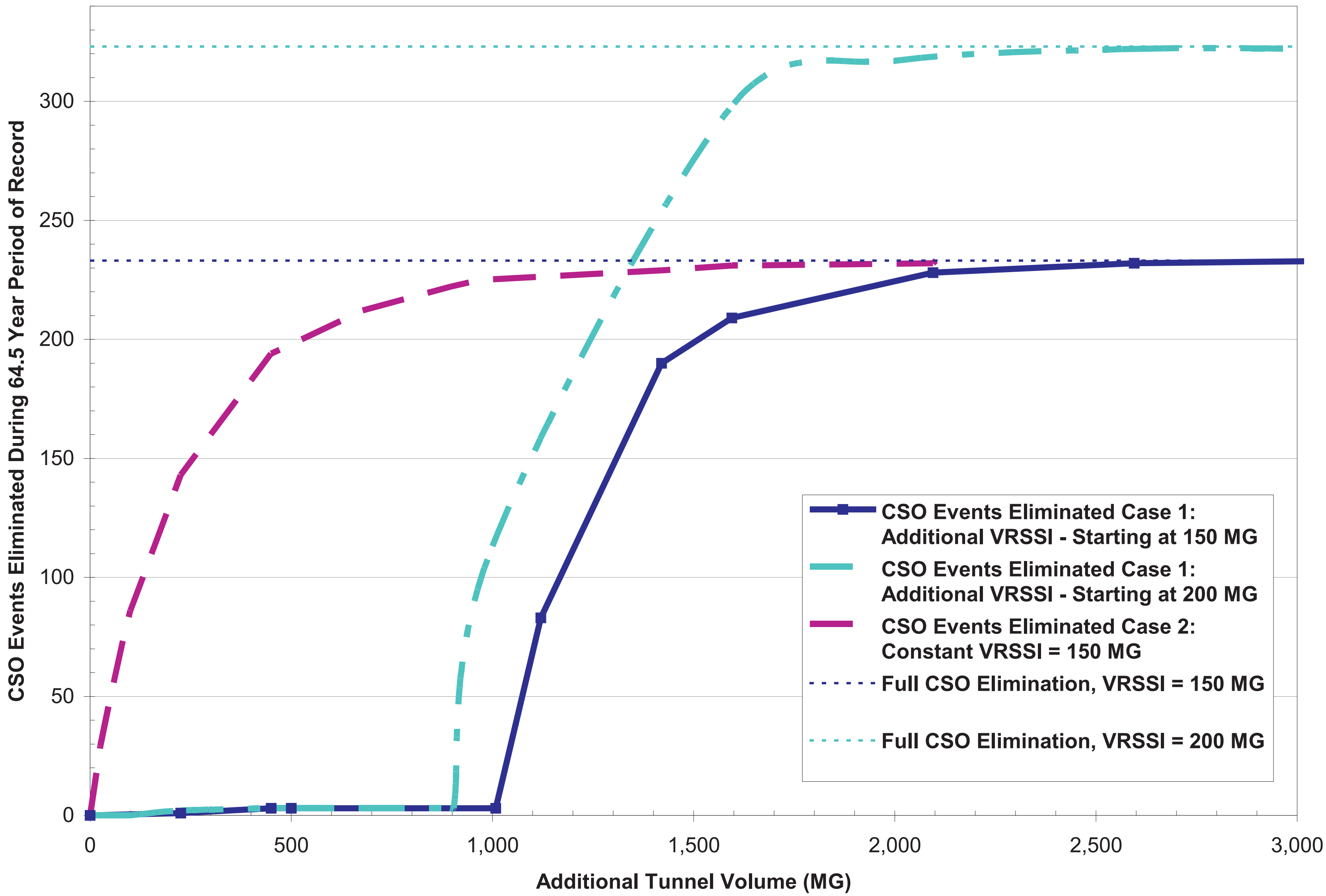

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FIGURE 3A-7

CSO EVENT PRODUCTION FUNCTIONS: DEEP TUNNEL STORAGE

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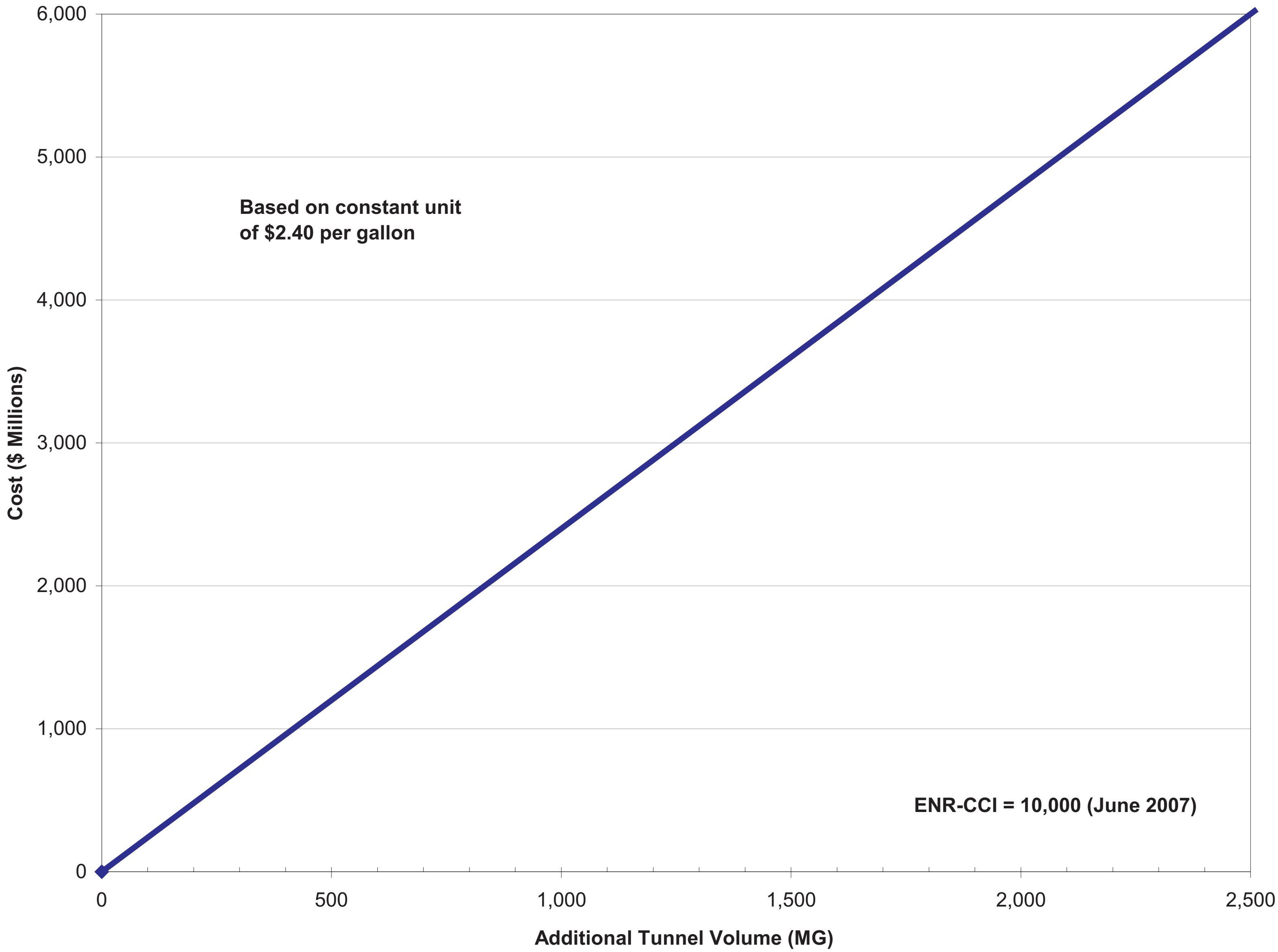

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FIGURE 3A-8
COST FUNCTION: DEEP TUNNEL STORAGE
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Cost Function Development

Cost information from completed MMSD projects was given the greatest weight. Cost estimates for proposed MMSD projects were also used. These MMSD sources are particularly relevant for larger tunnel volumes (over 100 MG). For small tunnel volumes, cost information from Indianapolis was used. The sources used were determined to be representative of typical tunnel costs, even though project details (such as geologic conditions) may differ somewhat.

The MMSD unit costs for deep tunnel storage are based on four sources of information (two are actual costs and two are estimated). The actual construction costs of the Crosstown tunnel completed in 1994 and the NWS Relief Sewer tunnel, completed in 2005, were used.

The cost values for the 32-foot diameter Crosstown tunnel were adjusted to account for lining (with a finished diameter of 30-foot). The values assume that a new tunnel would be located in good geologic strata similar to that of the Crosstown tunnel.

The NWS Relief Sewer tunnel has a 20-foot finished diameter. The tunnel is lined and is built in geologic conditions similar to the Crosstown tunnel.(1)

Estimated costs prepared for the Port Washington Road Relief Sewer project (now named the 27th Street ISS Extension) was also used to guide the tunnel unit cost.(2)

Two of the Port Washington Road alternatives use deep tunnel storage technology. The 27th Street alternative proposes a 28-foot diameter, 77 MG tunnel located in relatively good geologic conditions for tunnel construction. The Milwaukee River storage alternative proposes a 25-foot diameter, 53 MG tunnel located in geologic conditions that will make tunnel construction more difficult.(3)

Unit costs can rise dramatically if the strata are variable or highly fractured. This was the case with the North Shore tunnel, constructed in the 1980s and early 1990s where the unit costs more than doubled. The Crosstown rock strata, which are relatively tight and uniform, are better suited to construction. Rock excavated for the NWS Relief Sewer was also considered to be highly uniform; with allowances for grouting, four bids within 3% of each other were received.(4)

Table 3A-5 summarizes the unit costs from the four sources; these unit costs reflect the total cost per gallon of storage volume in terms of the 2007 total present worth. The unit cost values are sensitive to tunnel diameter and geologic conditions.

Additional cost information was based on estimates from the Indianapolis CSO Long Term Control Plan.(5)

These tunnel estimates are for tunnels ranging in size from 1.5 to 230 MG, with diameters ranging from 5- to 37-feet. Most of the 36 tunnels in this source are less than 100 MG; only eight estimates are for tunnels over 100 MG. The cost estimates from 2000 were scaled to 2007 values and account for the change from Indianapolis to Milwaukee.

TABLE 3A-5
UNIT COSTS FOR DEEP TUNNEL STORAGE BASED ON MMSD EXPERIENCE

Sources of Information	Unit cost $/gal	Finished Diameter (feet)	Geologic Conditions
Cross Town Tunnel	$2.25	30	Good
NWS Relief Sewer Tunnel	2.64	20	Good
Port Washington Rd Relief: 27th St - Deep Alternative	2.20	28	Good
Port Washington Rd Relief: Milwaukee River - Deep Alternative	2.49	25	Moderate to Poor
Unit Cost used for Cost Function	2.40		

Note:
All costs are estimated at an ENR-CCI of 10,000 (projected to be 2007).

Source: Brown and Caldwell, Port Washington Road Relief Sewer Project, Alternatives Analysis, Draft Memorandum, prepared for MMSD (2004)

Figure 3A-9 shows the unit costs graphically in terms of tunnel volume for both the Milwaukee and Indianapolis sources. The range of tunnel sizes used in the production functions (up to 2,600 MG) is much larger than any of the existing or future projects used as sources for the unit costs. It is likely that a large tunnel volume would be constructed incrementally, with each project adding approximately 50-100 MG. Therefore, it is suitable to use cost estimates in the 50-100 MG size range even though the production functions extend to 2,600 MG.

The unit cost for smaller tunnels is over $8.00 per gallon but decreases rapidly as the tunnel volume increases. For tunnel volumes over 100 MG, the constant unit cost of $2.40 per gallon was used for the cost function.

Cost Benefit Curves: Sanitary Sewer Overflow Indicator

A cost benefit curve is the combination of a production function and a cost function. The production function presents a benefit (such as SSO volume removed) as a function of a technology-specific parameter (such as additional tunnel volume). The cost function presents the cost as a function of the technology-specific parameter. By combining the two functions into a cost benefit curve, the dependence on a technology-specific parameter is eliminated. Consequently, many different technologies can be compared on the same graph.

The cost benefit curve shows the total present worth cost to achieve a desired benefit, such as the cost to remove SSO volume. The cost effectiveness of a technology is relatively good when the slope of the curve is relatively flat. Typically, the curves become steeper as the technology is applied to a greater degree; the steeper curves indicates relatively poor cost effectiveness.

Figure 3A-10 shows the cost benefit curves as additional volume of SSO is removed. Figure 3A-11 shows the cost benefit curves as additional SSO events are eliminated.

In Case 1 (with variable VRSSI), the slope of the SSO cost benefit curve is initially mild, but rises sharply in an attempt to remove the largest events. Case 2 (with constant VRSSI) is primarily intended to eliminate CSOs.

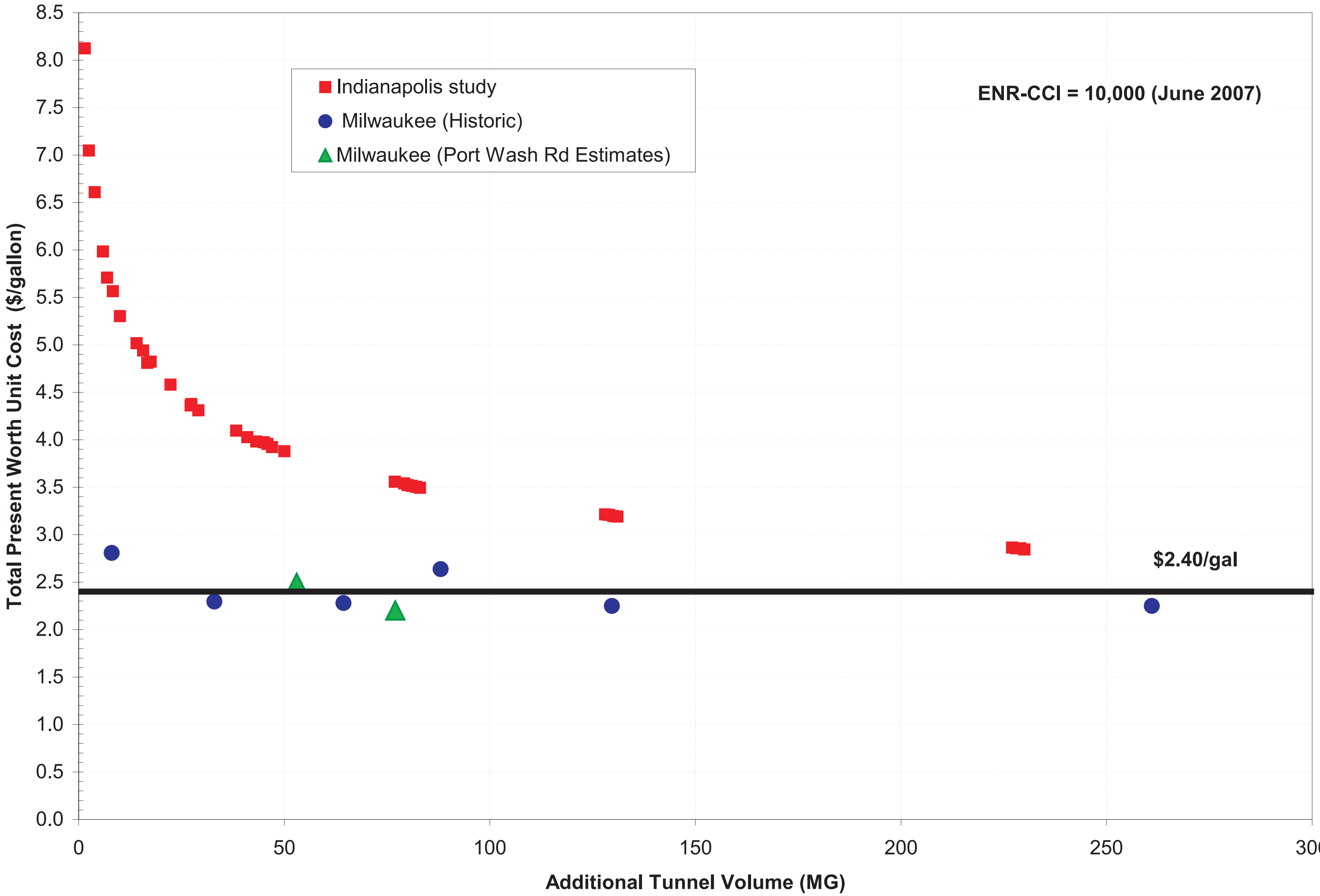

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FIGURE 3A-9
UNIT COST:
DEEP TUNNEL STORAGE
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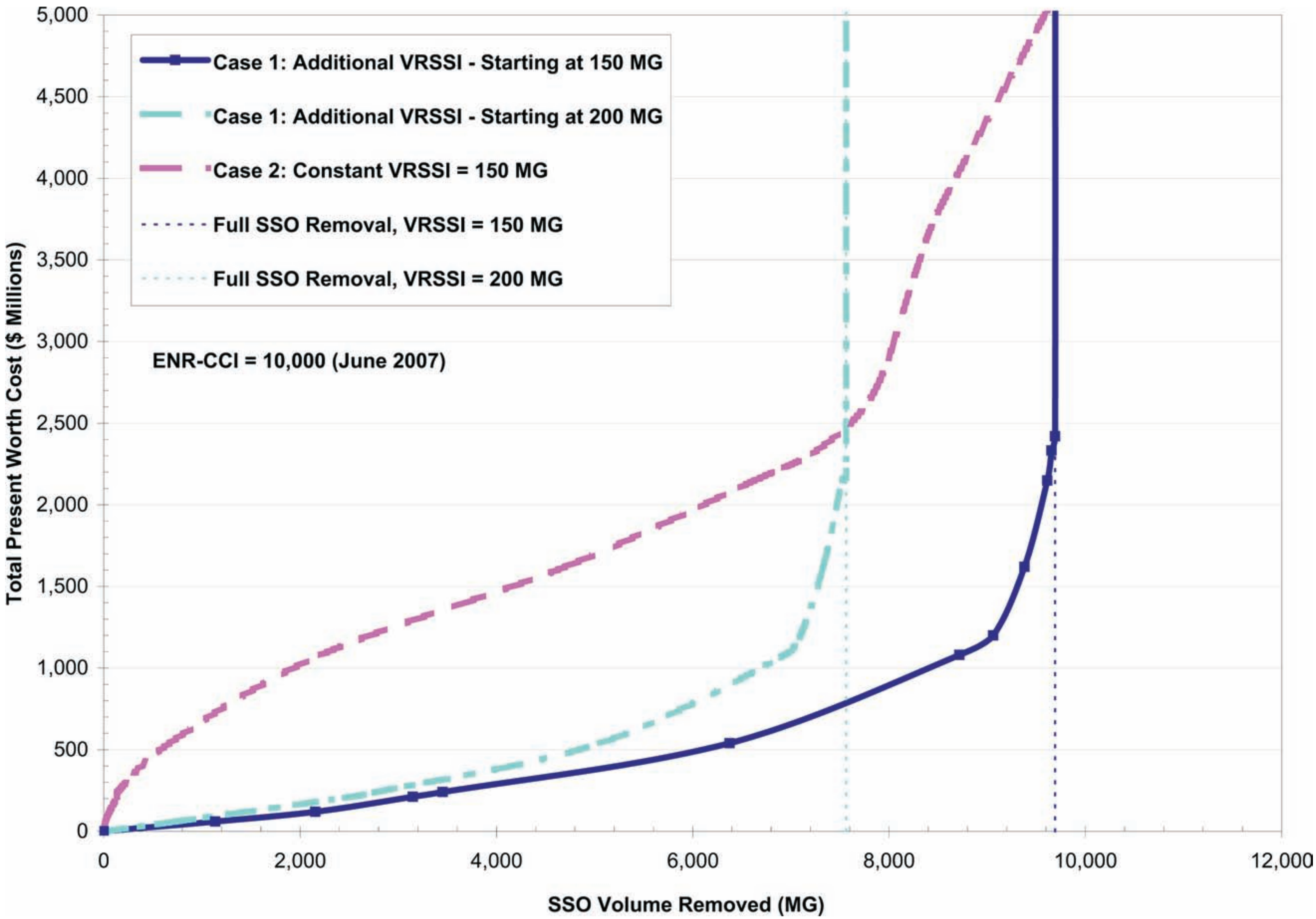

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FIGURE 3A-10
SSO VOLUME COST BENEFIT CURVES: DEEP TUNNEL STORAGE
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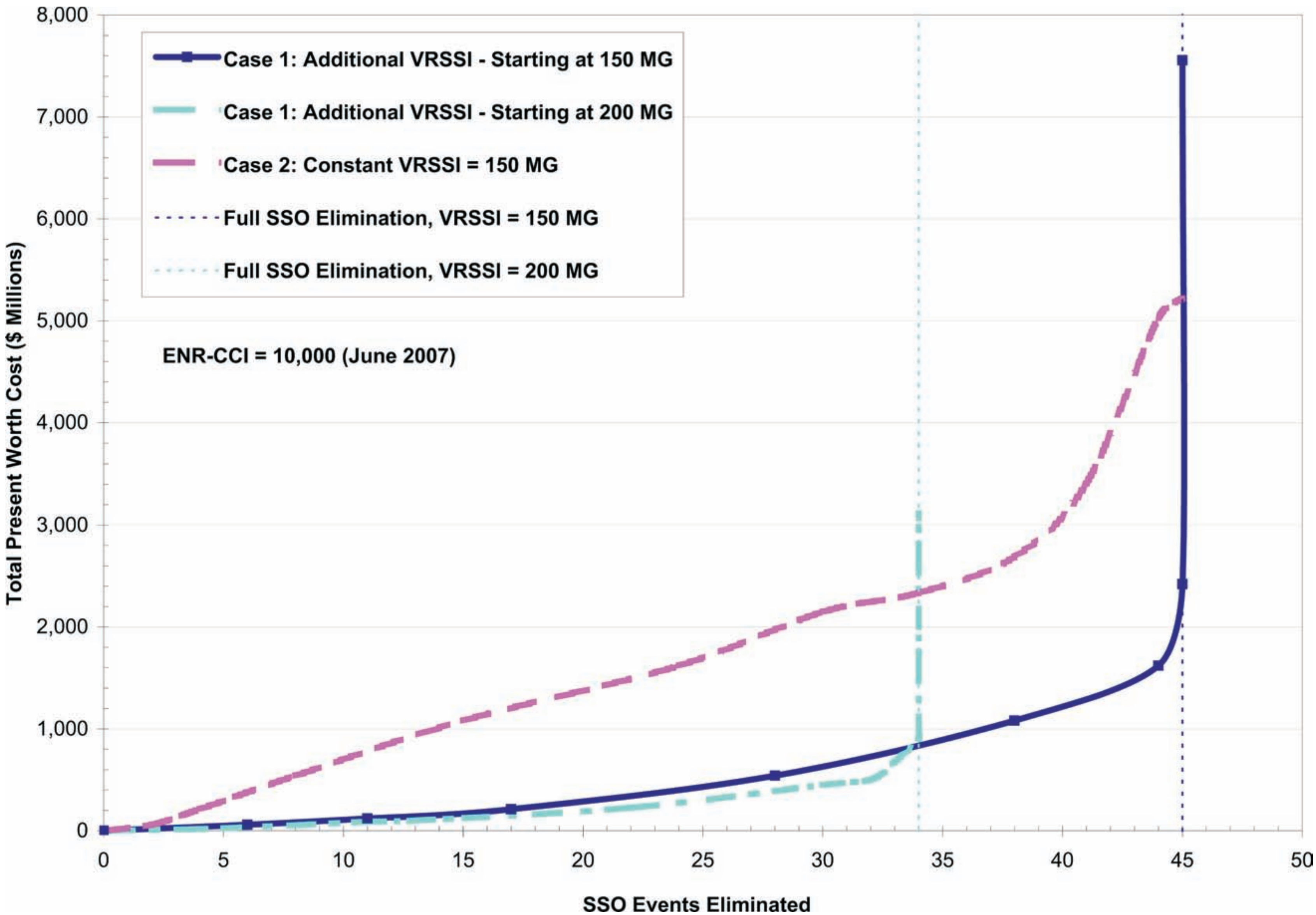

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FIGURE 3A-11
SSO EVENT COST BENEFIT CURVES: DEEP TUNNEL STORAGE
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Cost Benefit Curves: Combined Sewer Overflow Indicator

Figure 3A-12 shows the cost benefit curves as additional volume of CSO is removed. The CSO cost benefit curves for Case 1 and Case 2 have very similar shapes except for the initial value. The additional volume of VRSSI in Case 1 shifts the CSO curve upwards until all of the SSOs are eliminated, then the additional volume begins to provide benefit for CSO removal. Figure 3A-13 shows the cost benefit curves as additional CSO events are eliminated. When CSO removal is the objective, the Case 2 curves are relevant. The Case 1 curves are only shown for completeness.

Cost Benefit Curves: Total Overflow Volume Indicator

Figure 3A-14 shows the cost benefit curves for total overflow volume removed. The overflow volume in this figure does not distinguish between CSO and SSO volumes; consequently, the cost benefit curves show that Case 2 is more cost effective than Case 1.

Summary: Deep Tunnel Technology

Deep tunnel storage is a proven and effective technology to reduce both CSOs and SSOs. One of the key benefits of deep tunnel storage is that it provides surface water protection for the entire service area, much like additional wastewater treatment plant capacity but unlike regional technologies such as near surface storage. This is an advantage when the precipitation has a high degree of spatial variability, such as summer thunderstorm events.

3A.2.2 Technology: Cavern Storage

Introduction

The origin of the term "cavern storage" was within the context of MMSD's Milwaukee Water Pollution Abatement Program (MWPAP), which considered cavern storage in 1980 as an alternative approach to increase the capacity of the ISS if future needs mandated increased storage capacity. While not defining explicitly how it would be constructed, the concept was to create a series of intersecting tunnels (shape not defined) in a tight grid pattern in the vicinity of Miller Park and at an elevation slightly higher than the Crosstown Sewer such that as the Crosstown would fill, sewage would back up into the new cavern storage, and then exit by gravity as the sewage was pumped out at JIWWTP.(6)

For the MWPAP, the term cavern referred to a method of construction and a presumed shape, apart and different from that of a machine bored tunnel in rock. Cavern storage was to have been created with drill and blast methods in a horseshoe shape, but the concept never advanced beyond that point. Cavern storage never became a feature of the MWPAP's recommended plan. Certainly more conventional machine bored tunneling methods could have been considered.(7)

Drill and blast methods of construction in rock have been applied to tail tunnels, minor lengths of sewer connections, diversion structures, manholes and related appurtenances; however, these are minor facilities.(8) The ISS Pump Station is MMSD's only significant example of the cavern construction method. The excavation construction cost for the IPS was about $1,100,000/acrefoot in 1988 dollars.(9)

This value cannot be considered to be representative of drill and blast methods because of the small volume of the IPS and the highly fractured nature of the rock that required extensive rock bolting, steel mesh and shot-crete to stabilize the cavern surfaces.

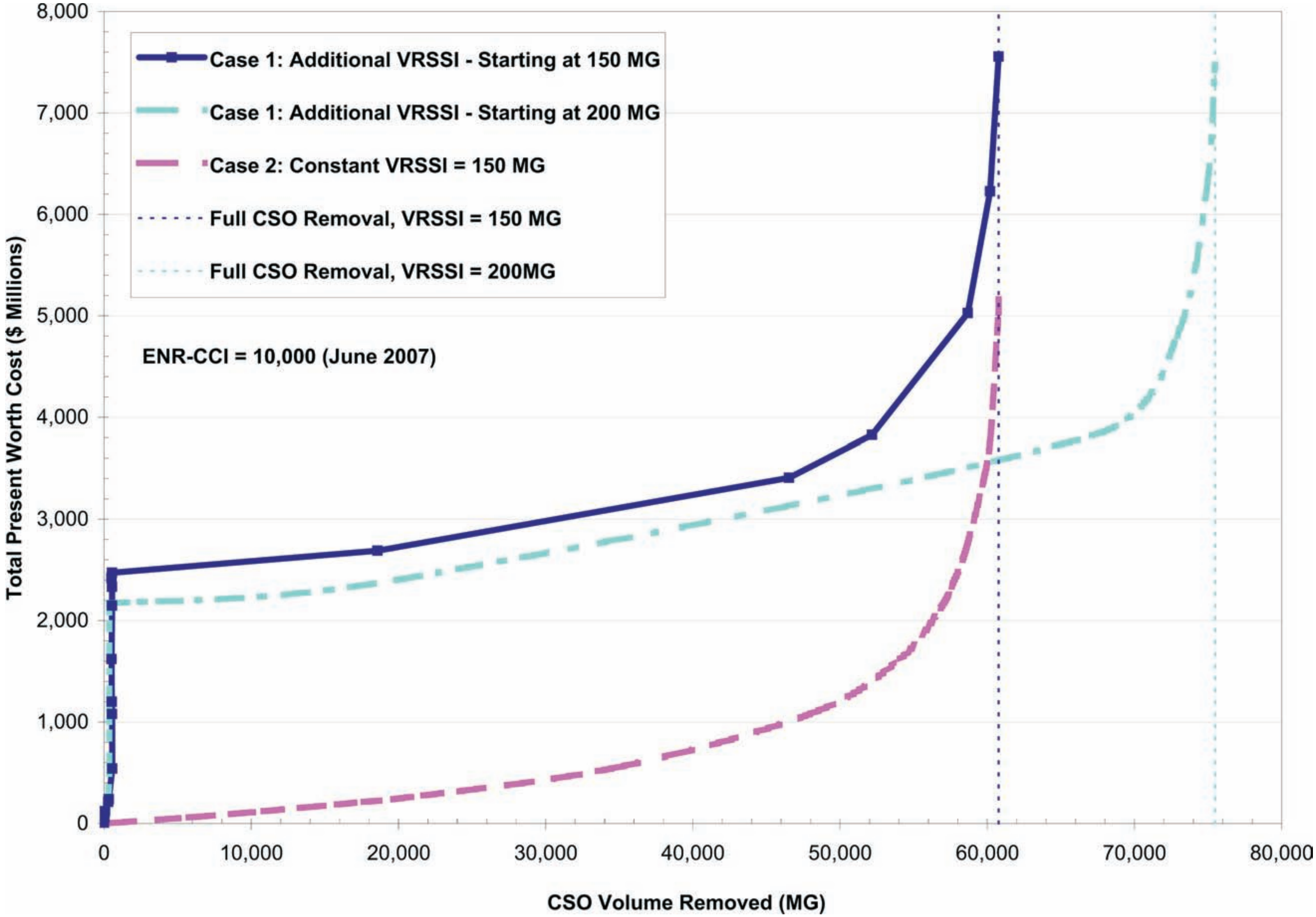

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FIGURE 3A-12

CSO VOLUME COST BENEFIT CURVES: DEEP TUNNEL STORAGE

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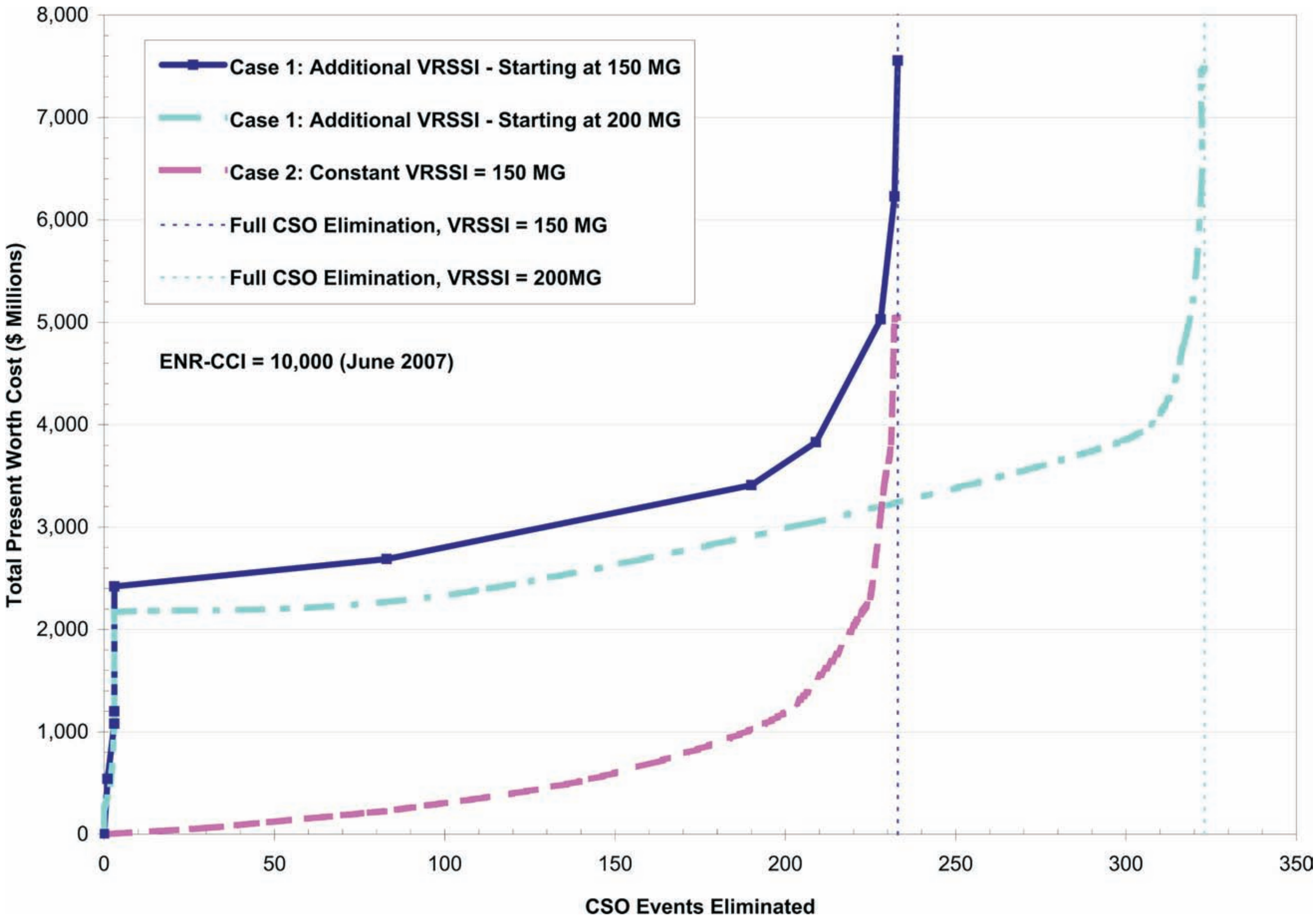

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FIGURE 3A-13

CSO EVENT COST BENEFIT CURVES: DEEP TUNNEL STORAGE

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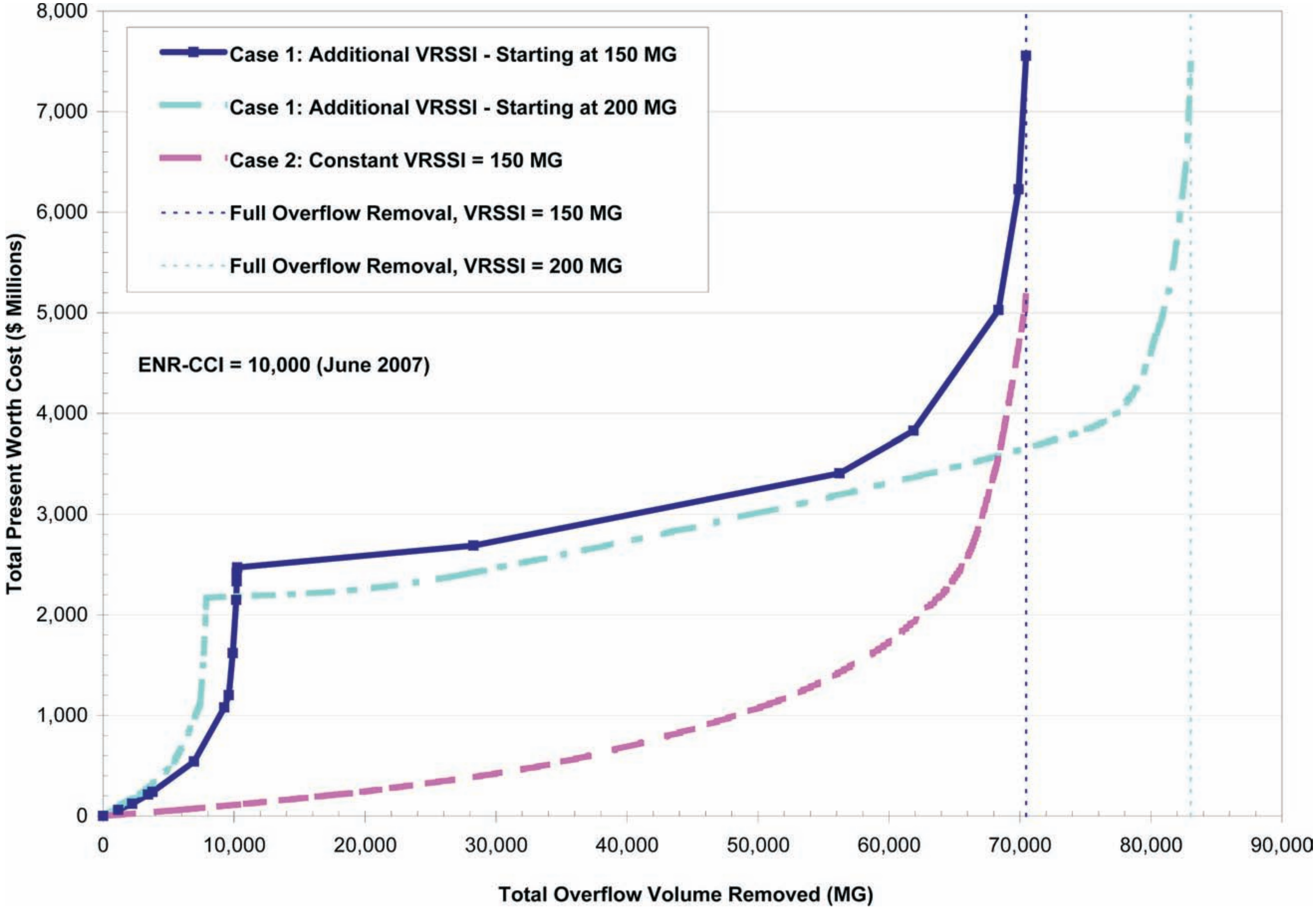

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FIGURE 3A-14

TOTAL OVERFLOW VOLUME COST BENEFIT CURVES: DEEP TUNNEL STORAGE

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The use of tunnel boring machines is commonplace; numerous contractors in the Milwaukee area have familiarity with these machines. If the traditional notion of cavern storage, as envisioned by MMSD in 1980, is expanded to include methods and shapes that are better suited to the contractors' experience and skill, and more compatible with environmental constraints, then the use of unit costs derived from machine bored tunnels provides a more realistic estimate for cavern storage.

Construction costs for cavern storage are, therefore, based on the tunnel storage values provided in Section 3A.2.1 for deep tunnel storage. The cavern storage option is not a separate technology, but a part of the deep tunnel technology given above.

Supplementary Source Information: Atlanta

Supplementary cost information is also available from a recently estimated cavern storage project in Atlanta, GA. The construction techniques are based on blasting to create a 10 MG box shaped cavern that is 100 ft below the surface and 700 ft long. Due to the nature of the site, specific geological conditions, and regional construction factors, it is difficult to determine how relevant this cost information is for the conditions in Milwaukee. Therefore, this information is only being provided as an additional reference.

The unit construction cost of the Atlanta cavern is approximately $1.70/gallon.(10)

Scaling from the Atlanta February 2005 value (ENR-CCI = 4607) to the Milwaukee 2007 value (ENR-CCI = 10,000), the cavern construction cost is $3.65/gallon.

Total cost includes contingency (25% of construction), technical services and non-construction cost (25% of construction and contingency), and O&M (3% of capital costs). Therefore, the total unit cost is $5.70/gallon. This is more than twice the deep tunnel unit cost.

This supplementary information is provided for reference. Because of tunnel construction experience locally in Milwaukee, the cost information for tunnel storage is more relevant than this supplementary information from Atlanta.

3A.2.3 Technology: Volume Reserved for Separate Sewer Inflow

Introduction

The VRSSI is the volume of the ISS reserved by closing the gates to the CSSA so that the remaining volume can store flow from the SSSA. The adjustment of the VRSSI makes a significant impact on the balance between CSO and SSO discharge. This analysis used a constant tunnel volume of 405 MG. Only the operational setting for the VRSSI was changed. This technology does not have a cost function because it is an operational adjustment of the existing system.

If the VRSSI is set to zero, then the tunnel is operated in a "first-in-first-out" mode, which will provide the highest protection against CSOs but little protection for SSOs. If the VRSSI is set equal to the tunnel volume of 405 MG, then no flow from the CSSA is allowed into the ISS; the entire tunnel volume is used to minimize SSOs.

Experience

The ISS has been operated using initial VRSSI values between 40 and 200 MG. As of August 26, 2005, the default value of VRSSI is 250 MG, which is the starting VRSSI in any wet weather event. The VRSSI may be adjusted during an event to reflect specific conditions of the event such as tunnel inflow rates and available weather predictions.

Combined Sewer Overflow and Separate Sewer Overflow Frequency

Figure 3A-15 shows the trade off between CSO and SSO events in response to changing the VRSSI. As VRSSI increases, the number of CSOs increases and the number of SSOs decreases. The figure shows the total number of modeled events during the 64.5-year period of record for existing and future conditions.

The simulated recurrence interval of SSOs is shown in Figure 3A-16A for both Existing 2000 and 2020 Baseline conditions. The recurrence interval is the average number of years between SSO events. As the VRSSI increases, more of the tunnel volume is available for separate sewer inflow and, correspondingly, the SSOs decrease; consequently, the average number of years between events increases.

Figure 3A-16B shows the average number of CSO events per year. As the VRSSI increases, less tunnel volume is available for combined sewer inflow and therefore, the number of CSOs increases. The existing and future conditions are very similar; however, the 2020 Baseline results are slightly less than the Existing 2000 results. The CSOs are somewhat reduced in the 2020 Baseline condition because the upgraded harbor siphons convey more flow directly from the CSSA to JIWWTP.

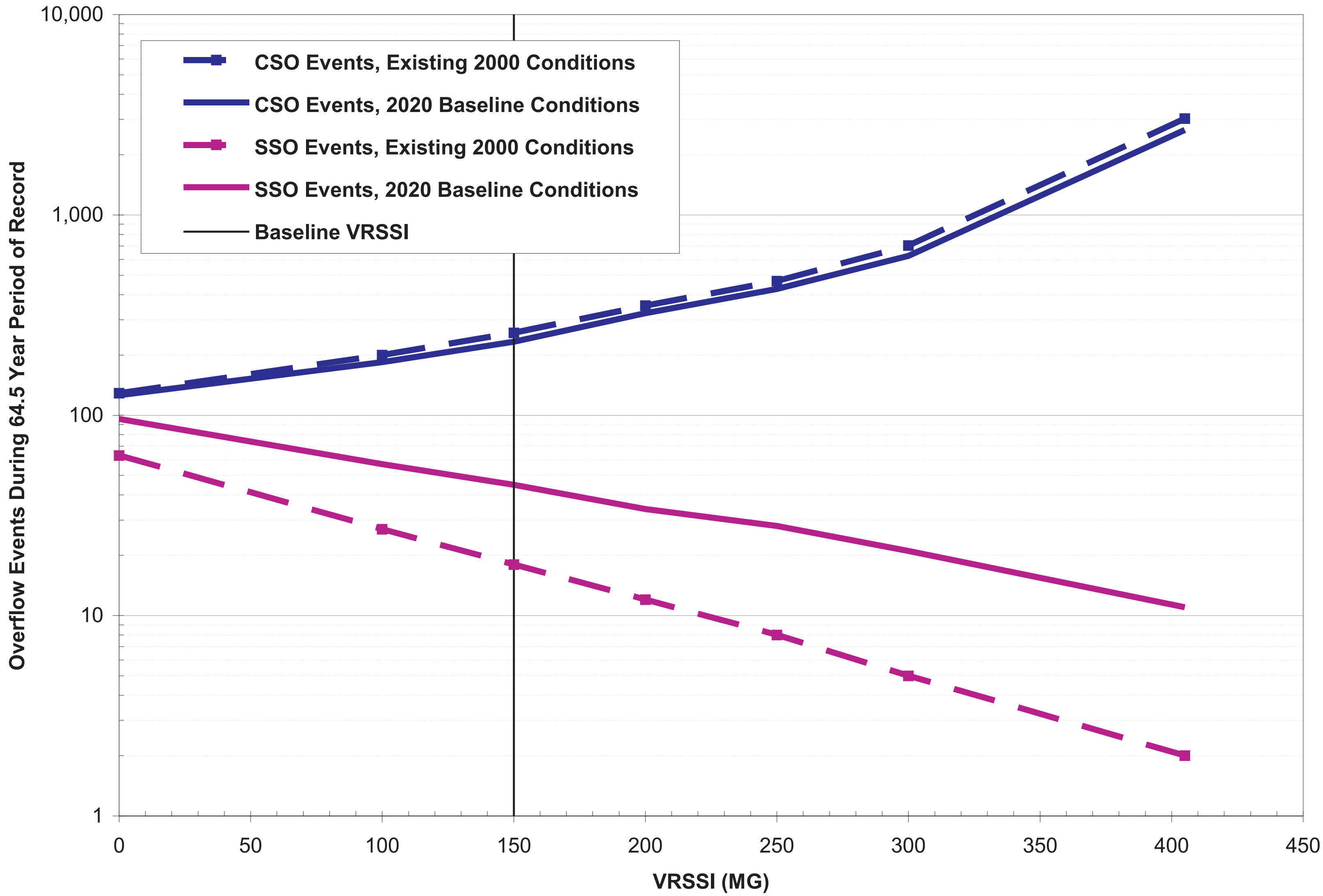

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FIGURE 3A-15

SIMULATED CSO AND SSO EVENTS DURING THE PERIOD OF RECORD AS A FUNCTION OF VRSSI

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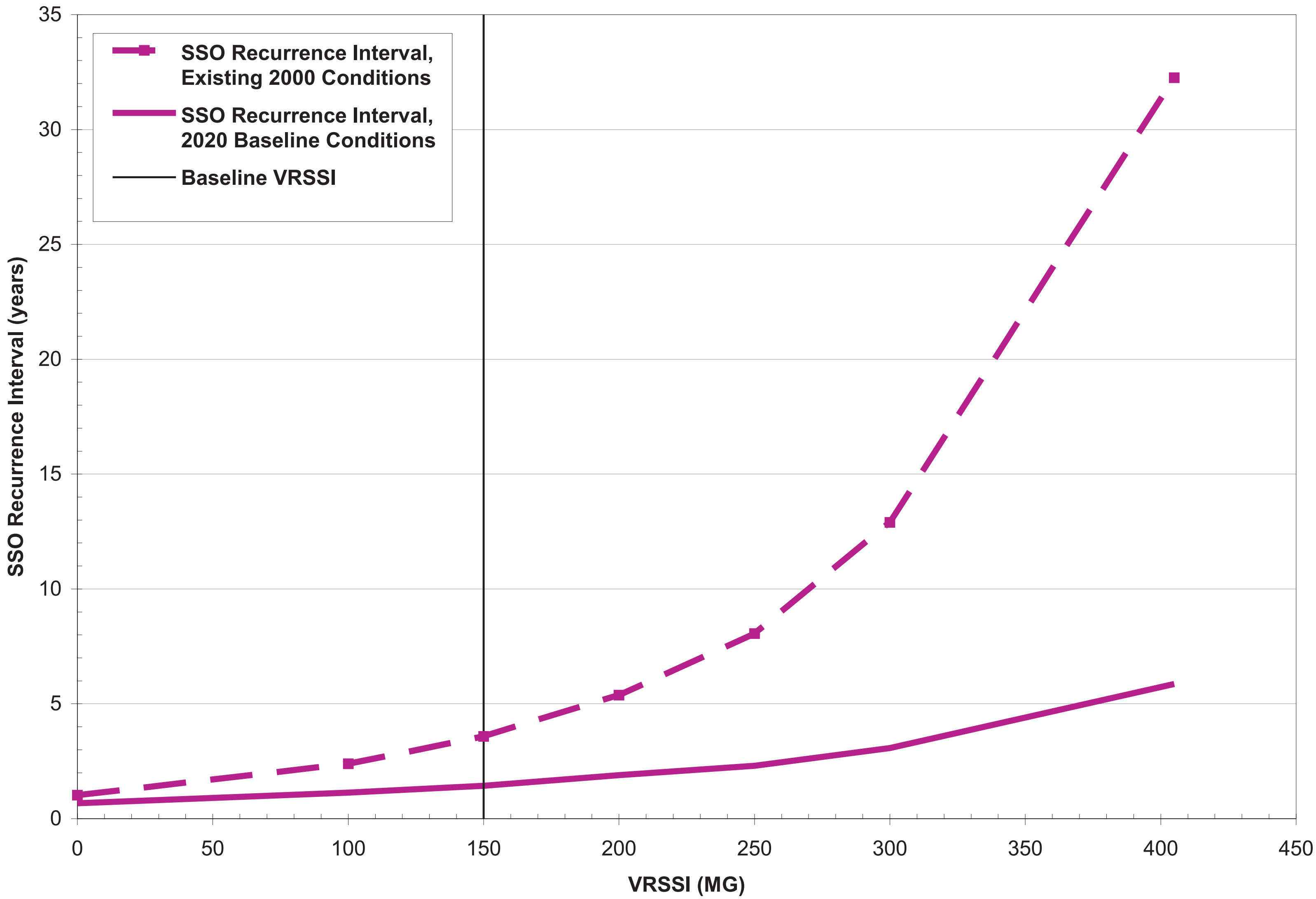

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FIGURE 3A-16A

SIMULATED SSO RECURRENCE INTERVAL AS A FUNCTION OF VRSSI

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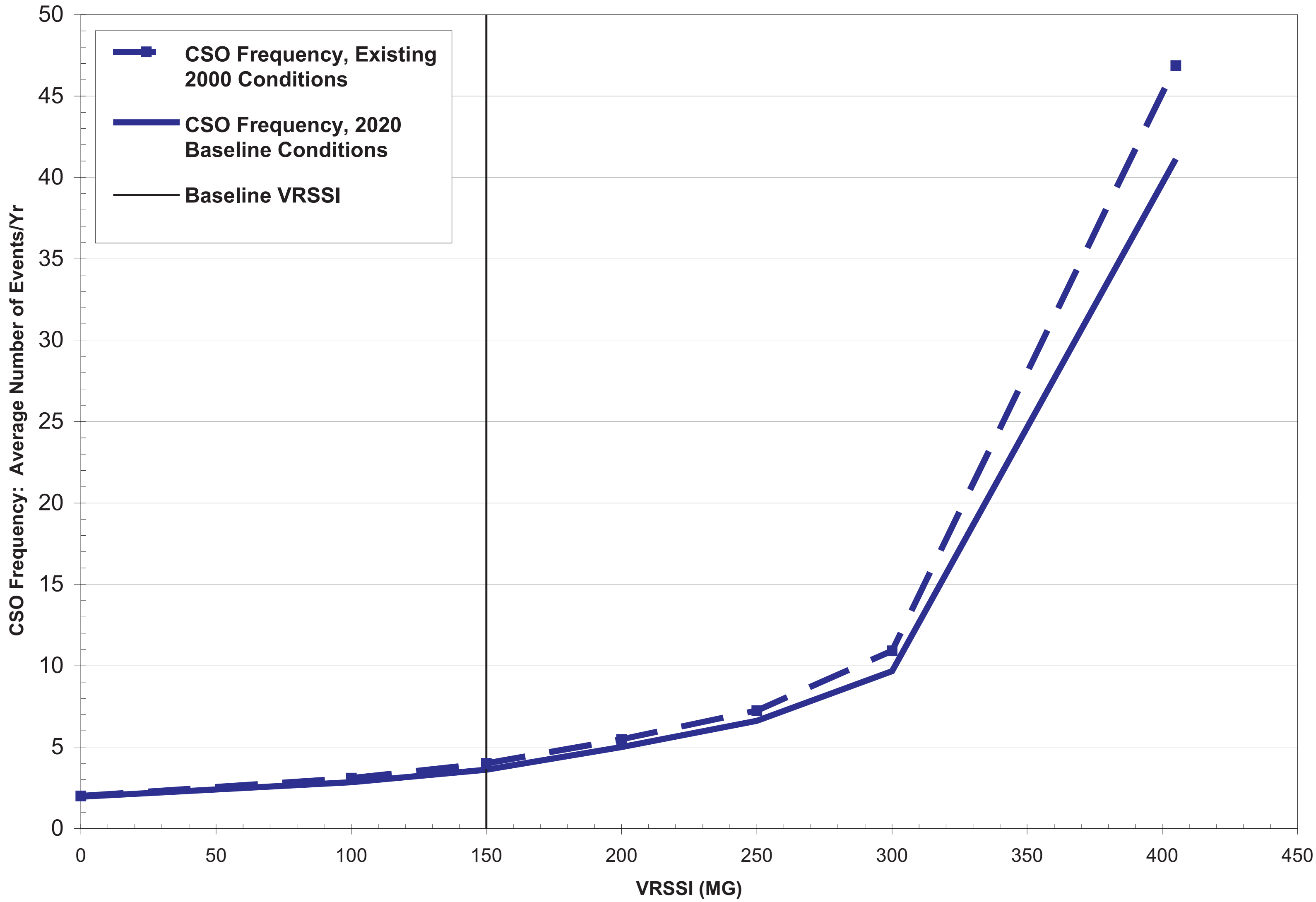

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FIGURE 3A-16B
SIMULATED CSO FREQUENCY AS A FUNCTION OF VRSSI
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The CSO frequency is the average number of CSO events per year. With VRSSI equal to 150 MG, the average number of CSOs is 3.6 per year for the 2020 Baseline conditions. Figure 3A-17 shows the simulated number of CSO events for each year during the 64.5-year period of record; the maximum number of CSOs was nine events during the simulation of the year 2000. This is a simulated result using the constant value for VRSSI in each simulation; these simulations do not attempt to reconstruct the actual measured number of CSOs during 2000 in which the VRSSI was varied by the operators.

The current Wisconsin Pollutant Discharge Elimination System (WPDES) permit limit is six CSO events per year. The simulation results are in excess of the permit limit for three years during the simulated period of record. The actual performance of the system has not violated the permit limit for CSOs to date. Figure 3A-18 shows the average and the maximum number of CSOs per year as a function of the modeled (fixed) VRSSI. For the maximum number of CSOs to be equal to six, the VRSSI would need to be zero for 2000 conditions. For 2020 conditions, there were seven simulated CSO events with VRSSI equal to zero.

Figure 3A-19 shows the average annual volume of overflows for both CSOs and SSOs. As the VRSSI increases, the CSO volume increases and the SSO volume decreases. The total overflow volume - the sum of the CSO and the SSO volumes - also increases with the VRSSI. The lowest value of total overflow volume is when VRSSI is equal to 0; however, this does not distinguish between CSOs and SSOs.

Production Functions

The production functions for VRSSI show the trade off between CSOs and SSOs; the baseline case is VRSSI equal to 150 MG. The other results are compared relative to the baseline case.

When the VRSSI is above 150 MG, the volume of SSO removed becomes greater while the volume of CSO removed decreases. In other words, there is more CSO volume. Figure 3A-20 shows the volume production functions. If the full tunnel is dedicated to the SSSA (VRSSI = 405 MG) then 78% of the total SSO volume is removed. At the same time, the CSO volume will become three times larger.

Using the "first-in-first-out" operational strategy (no VRSSI), the SSO volume is approximately twice as large as the baseline case and the CSO volume is approximately half.

Figure 3A-21 shows the relationship between varying VRSSI and elimination of CSO events. Greater values of VRSSI improve the SSO elimination, while CSO elimination is hindered. When the VRSSI is fixed at 200 MG, there are 11 SSOs eliminated, but 90 additional CSO events.

Cost Function

Because the VRSSI is an operational variable, there are no construction or O&M costs associated with this technology. Therefore, there is no cost function.

Summary: Volume Reserved for Separate Sewer Inflow

The VRSSI is an operational parameter that adjusts the amount of tunnel volume reserved for separate sewer flow. Consequently, the VRSSI can change the balance between SSO and CSO discharges; the VRSSI can be adjusted to improve one at the expense of the other. Because there is no capital cost associated with this technology, it is not possible to compare it to other technologies using a cost benefit curve.

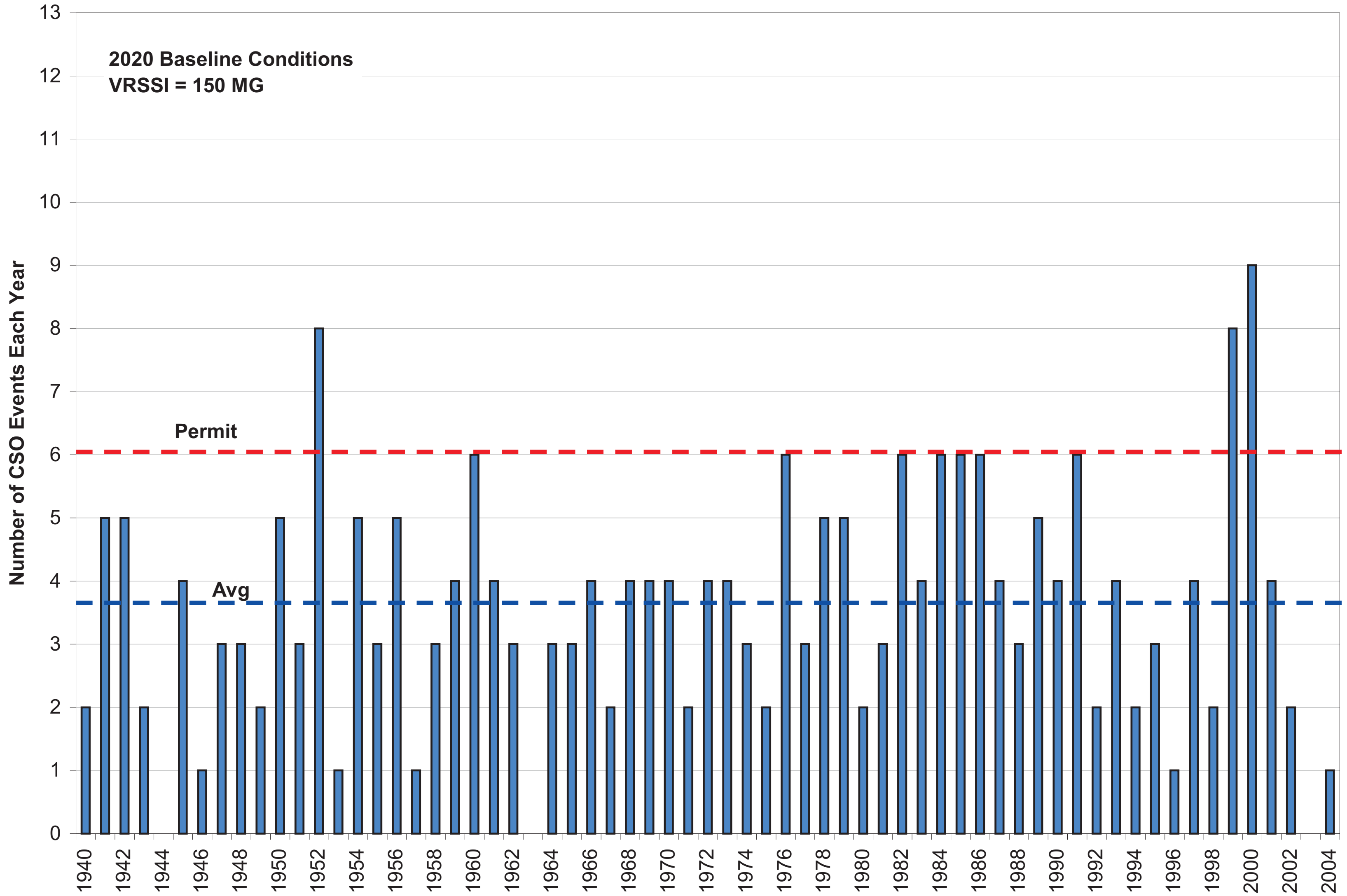

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FIGURE 3A-17

SIMULATED NUMBER OF CSOs EACH YEAR DURING PERIOD OF RECORD FOR 2020 BASELINE CONDITION WITH FIXED VRSSI OF 150 MG

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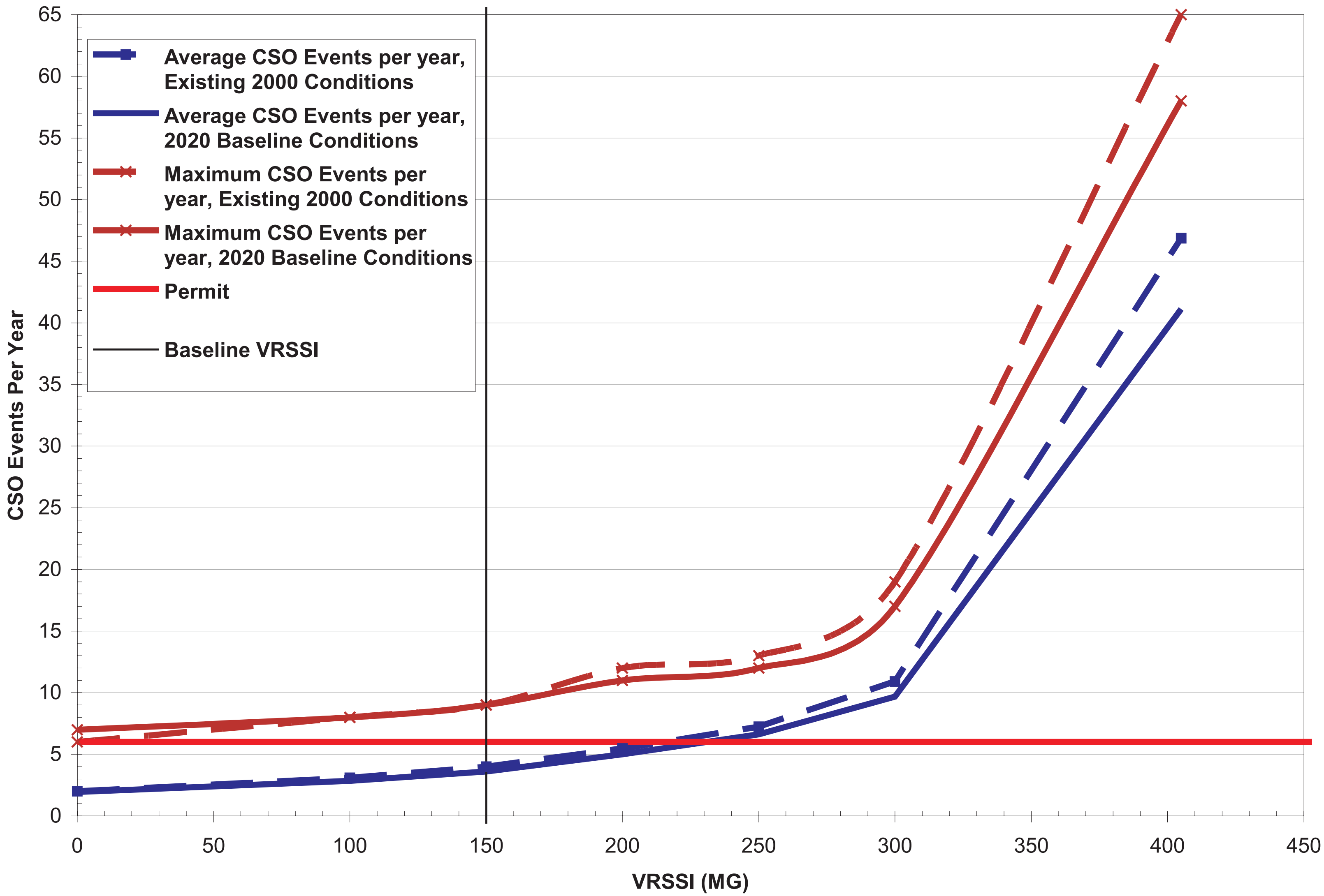

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FIGURE 3A-18
AVERAGE AND MAXIMUM NUMBER OF CSOs EACH YEAR DURING THE PERIOD OF RECORD
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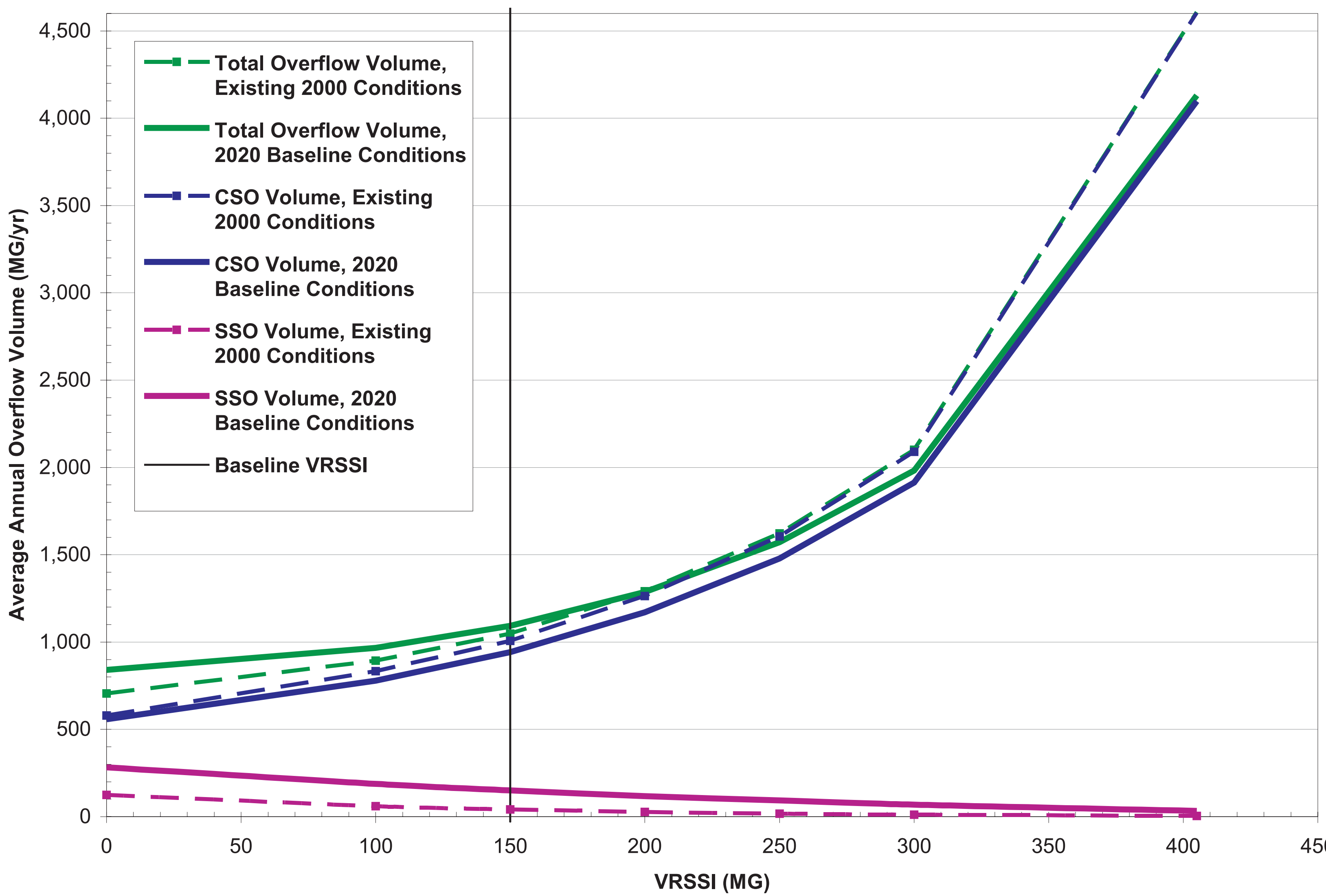

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FIGURE 3A-19
SIMULATED AVERAGE ANNUAL OVERFLOW VOLUME AS A FUNCTION OF VRSSI
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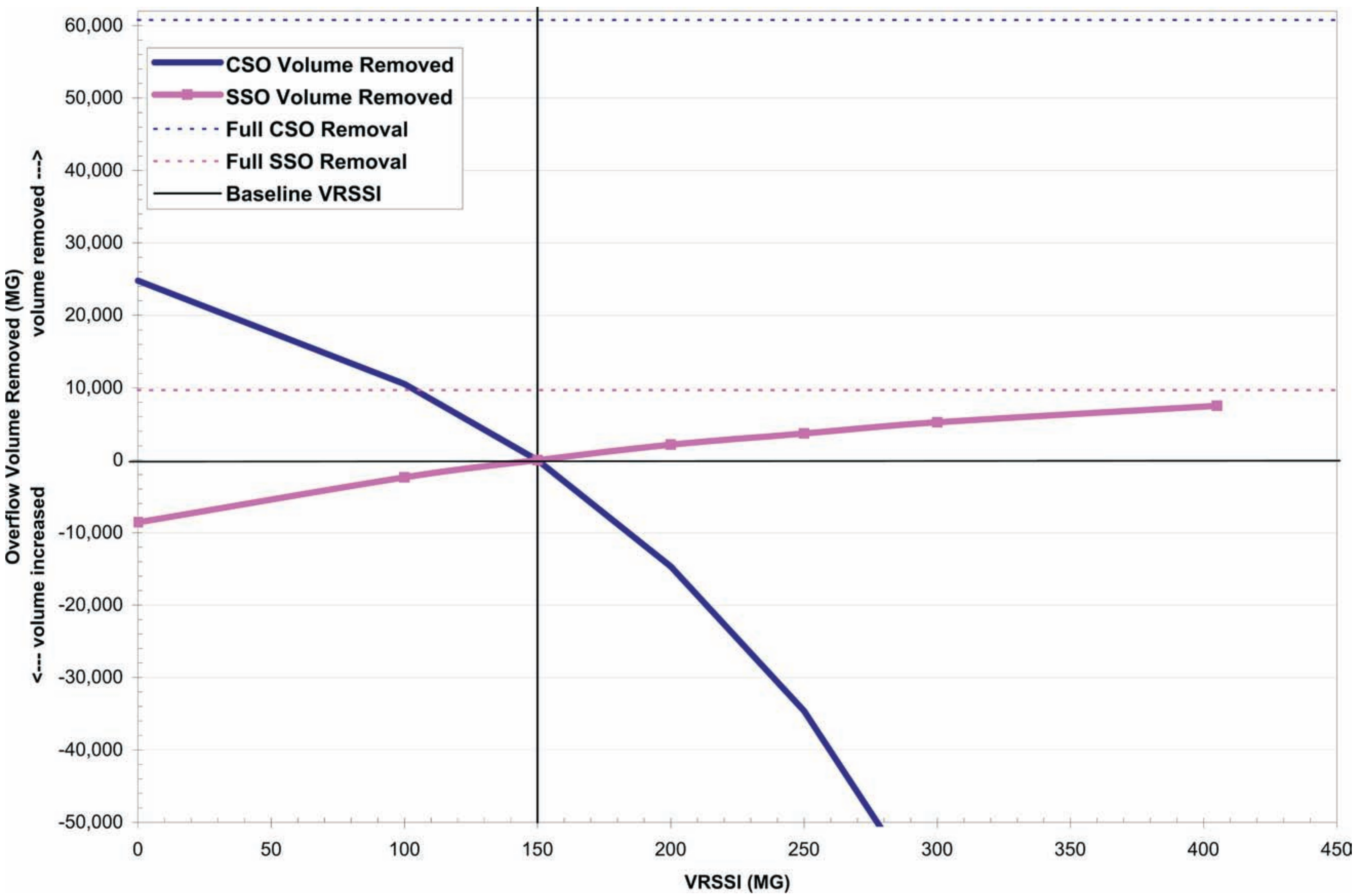

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FIGURE 3A-20
OVERFLOW VOLUME PRODUCTION FUNCTIONS: VRSSI FOR 2020 BASELINE CONDITIONS
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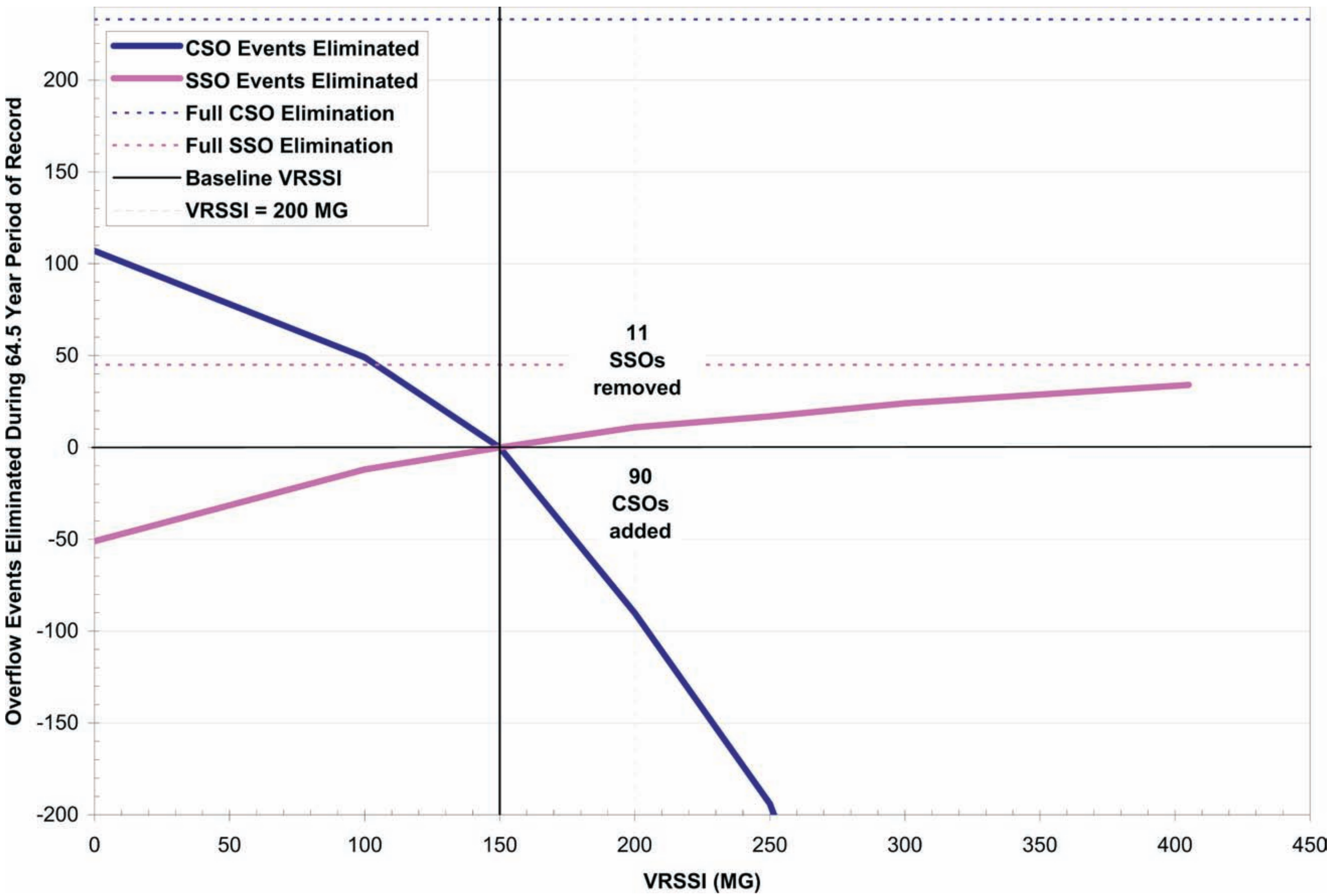

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FIGURE 3A-21

OVERFLOW EVENT PRODUCTION FUNCTIONS: VRSSI FOR 2020 BASELINE CONDITIONS

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Table 3A-6 is a summary of the results for VRSSI. Based on the MACRO simulation results using VRSSI equal to 150 MGD, the Existing 2000 system has an average CSO frequency of 4.0 events per year and the 2020 Baseline system has an average CSO frequency of 3.6 events per year.

TABLE 3A-6
SUMMARY OF VRSSI RESULTS FOR LEVEL OF PROTECTION

	SSO LOP Recurrence Interval (yr)		CSO events/yr	
VRSSI (MG)	**Existing 2000**	**2020 Baseline**	**Existing 2000**	**2020 Baseline**
0	1.0	0.7	2.0	2.0
150	3.6	1.4	4.0	3.6
200	5.4	1.9	5.5	5.0
250	8.1	2.3	7.2	6.6
405	32.3	5.9	46.9	41.1

CSO = Combined Sewer Overflow
MG = Million Gallons
VRSSI = Volume Reserved for Separate Sewer Inflow
LOP = Level of Protection
SSO = Sanitary Sewer Overflow

Source: MACRO simulation results

If the tunnel operations are changed to reduce CSOs (with no VRSSI), the average number of CSO events per year could be as low as 2.0 for Existing 2000 and 2020 Baseline conditions.

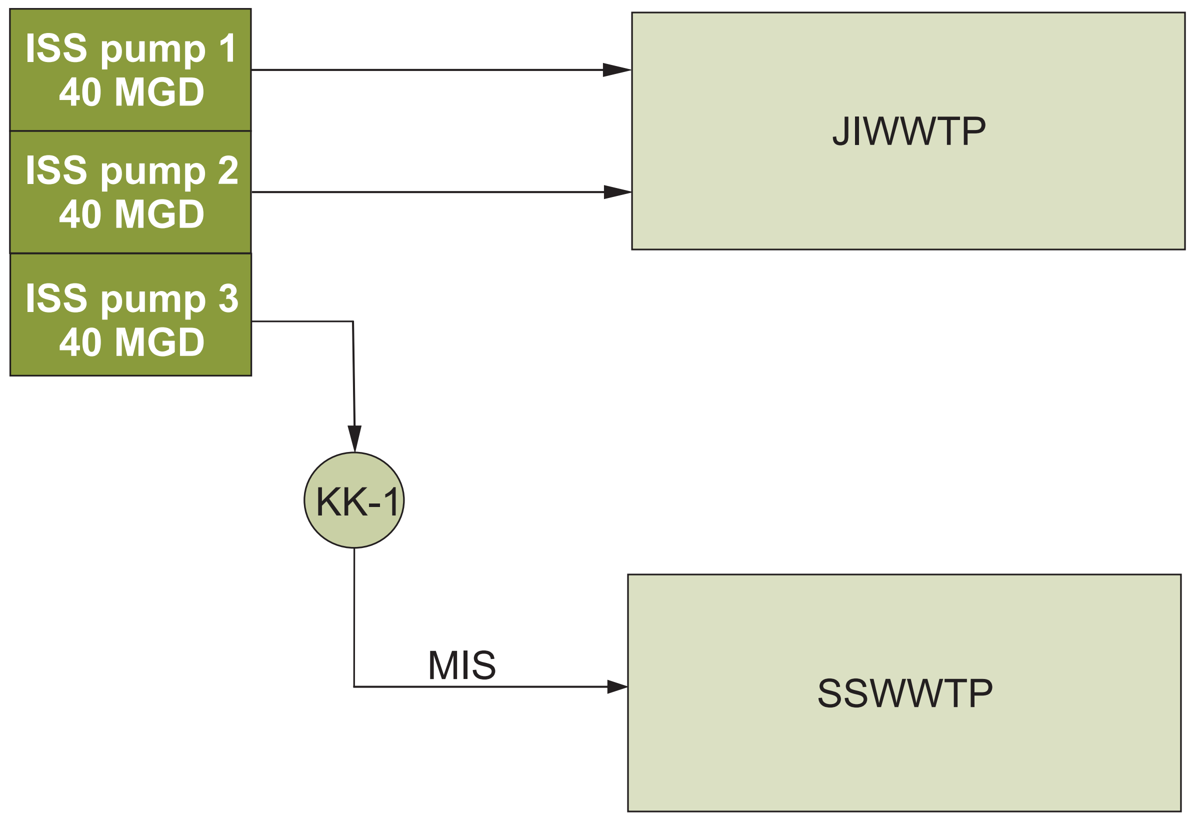
The average recurrence interval of SSO events is approximately 3.6 years for the Existing 2000 system and 1.4 years for the 2020 Baseline system for the baseline case with VRSSI equal to 150 MG. If the tunnel operations were changed to reduce SSOs by reserving the full tunnel volume for separate area flow with VRSSI equal to 405 MG, the SSO recurrence interval could be as large as 32 years for the Existing 2000 system and 6 years for the 2020 Baseline system.

3A.2.4 Technology: Inline Storage System Pumping

Introduction

The ISS Pump Station at JIWWTP has three pumps, each with a rated capacity of 50 MGD but with actual capacity of approximately 40 MGD based upon pumping records available in 2005. Flow can be pumped to both JIWWTP and SSWWTP. The flow is pumped to SSWWTP via a force main to a diversion chamber located at South 6th Street and West Oklahoma Avenue. From this point, the flow enters the 96-inch MIS to SSWWTP. Figure 3A-22 is a schematic of the pumping relationships. Typically two pumps are directed to JIWWTP and one pump to SSWWTP, but it is possible to direct all three pumps to JIWWTP.

Gravity flow to each plant is processed first, up to the treatment capacities of the plants. The ISS pumps supply flow to the treatment plants only when there is treatment capacity that is not being utilized by gravity flow.



ISS = Inline Storage System
MIS = Metropolitan Interceptor Sewers
JIWWTP = Jones Island Wastewater Treatment Plant
SSWWTP = South Shore Wastewater Treatment Plant
KK-1 = The drop shaft to the ISS near 6th and Cleveland

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FIGURE 3A-22
SCHEMATIC OF THE ISS PUMPED FLOW TO THE WASTEWATER TREATMENT PLANTS
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During the peak of a wet weather event, excess flow that cannot be processed by the treatment plants is stored in the ISS. When the full treatment capacity at either plant is not fully utilized by gravity flow, the ISS is pumped to empty the tunnel and use the available treatment capacity. In the model the pumped flow rate is variable, making up the difference between the plant capacity and the gravity flow as long as there is volume in the ISS. The pumped flow can increase up to the pumping limits which are 80 MGD to JIWWTP and 40 MGD to SSWWTP.

Existing Pumps: Sensitivity to Operational Choice

Using the three existing pumps, the baseline operational setup directs two pumps to JIWWTP and one pump to SSWWTP. As an alternative, all three pumps can be directed to JIWWTP. The impact of this operational choice is presented in Table 3A-7 for 2020 Baseline conditions. The simulated number of SSO events decreases from 45 to 37 events; thus the average annual number of SSOs decreases from 0.70 to 0.57 events per year. The simulated number of CSO events increases slightly from 233 to 240 CSOs during the 64.5-year period of record so that average number of CSO increases from 3.6 to 3.7 events per year.

TABLE 3A-7
SENSITIVITY TO OPERATIONAL CHANGE: ALL THREE PUMPS TO JIWWTP

ISS Pumping to JIWWTP	ISS Pumping to SSWWTP	Average SSO Volume per Year (MG/yr)	Average # of SSO Events/year	Average CSO Volume per Year (MG/yr)	Average # of CSO Events/Year
2 pumps 80 MGD	1 pump 40 MGD	151	0.70	942	3.6
3 pumps 120 MGD	0	106	0.57	928	3.7

ISS = Inline Storage System
JIWWTP = Jones Island Wastewater Treatment Plant
MGD = Million Gallons per Day
CSO = Combined Sewer Overflow
SSWWTP = South Shore Wastewater Treatment Plant

Source: 2020 Baseline conditions, MACRO simulation results

The pumping capacity can be used sooner after the peak of the event when all three pumps are directed to JIWWTP and the ISS can be dewatered more quickly. Consequently, the SSO volume can be reduced by 30% and the number of SSO events can be reduced by 18%. The CSO volume is reduced 2%. The number of CSO events increases slightly even though the total CSO volume reduced. The effectiveness of pumping is sensitive to the timing of the peak flows to JIWWTP and SSWWTP. The MACRO model does not simulate the timing of peak flows as accurately as the Model of Urban Sewers (MOUSE) model. Therefore, these results need to be interpreted as screening results to define the general cause-and-effect relationship for pumping and no cost benefit curves were developed (originally identified as the deep tunnel pumping modes and methods technology in Chapter 2, *Technology/Indicator Analysis*). The analysis can be refined by using the MOUSE model.

Additional Pumping Facilities: Design Factors

The remainder of this section evaluates the benefits and costs of increasing the pumping capacity of the existing facilities. Production functions are developed for additional pumping to JIWWTP and to SSWWTP. Pumping to JIWWTP is more cost effective and beneficial than pumping to SSWWTP for two reasons: the location of the pump station and the opportunity time available for pumping to JIWWTP. These design factors are discussed first; then, the production functions are presented.

Pumping to JIWWTP is preferred because the pump station is located at JIWWTP. Additional pumping to SSWWTP requires the construction of a large force main from the ISS Pump Station to SSWWTP. Without a force main, the opportunity to pump to SSWWTP is limited.

With the existing configuration, the ISS pump to SSWWTP is directed to the MIS near KK-1 (the site of an ISS drop shaft near South 6th Street and West Cleveland Avenue); from that point it continues on to SSWWTP by gravity. Consequently, the pump cannot be used while the ISS gates are open; otherwise, the flow would re-enter the ISS. A new force main would allow the ISS to be pumped directly to SSWWTP whenever the plant has reserve capacity and would not be limited to the times when the ISS gates are closed.

The second advantage to pumping to JIWWTP is that the plant has a high peak capacity but is only able to operate at that peak rate for a short period of time. The gravity flow to JIWWTP is primarily from the CSSA while the flow to SSWWTP is from the SSSA. The characteristic hydrologic response of the CSSA is “flashy” (high flow rates for short periods of time). Consequently, the peak flow to JIWWTP has a short duration.

In the simulations, the treatment capacities of the plants are not changed, only the ISS pumping limits are increased. The benefit of additional ISS pumping depends on the availability of treatment capacity while the ISS has volume to be pumped. Additional pump capacity can be particularly helpful for back-to-back storms; the additional capacity can pump the tunnel down before the next event arrives. However, if treatment plant capacity is not available, additional pump capacity will be useless.

Figure 3A-23 shows the flow at JIWWTP during the simulated March 1960 event. Two cases are shown; in the baseline case, the ISS pumps to JIWWTP are limited to 80 MGD. The flow at JIWWTP reflects the flashy character of the CSSA. The treatment plant operates at the peak capacity of 390 MGD during a few brief peaks. By increasing the pump limit to JIWWTP to 300 MGD, the plant can operate at full capacity for an extended period of time so that the ISS can be pumped out faster.

The SSWWTP, on the other hand, operates at peak capacity for a long period of time due to the slower hydrologic response of the SSSA. Therefore the “opportunity time” for pumping to SSWWTP is not a significant factor. When the gravity flow at SSWWTP finally recedes below the peak plant capacity, much of the ISS volume has already been pumped to JIWWTP.

Figure 3A-24 is an example of the flow at SSWWTP for the simulated March 1960 event. Even though the model was configured with a pump limit of 300 MGD to SSWWTP, the pump only turned on briefly for three hours early in the event. After the peak, the ISS was empty before the gravity flow at SSWWTP receded below the treatment capacity. Consequently, there was little time available to use the pumping capacity to SSWWTP.

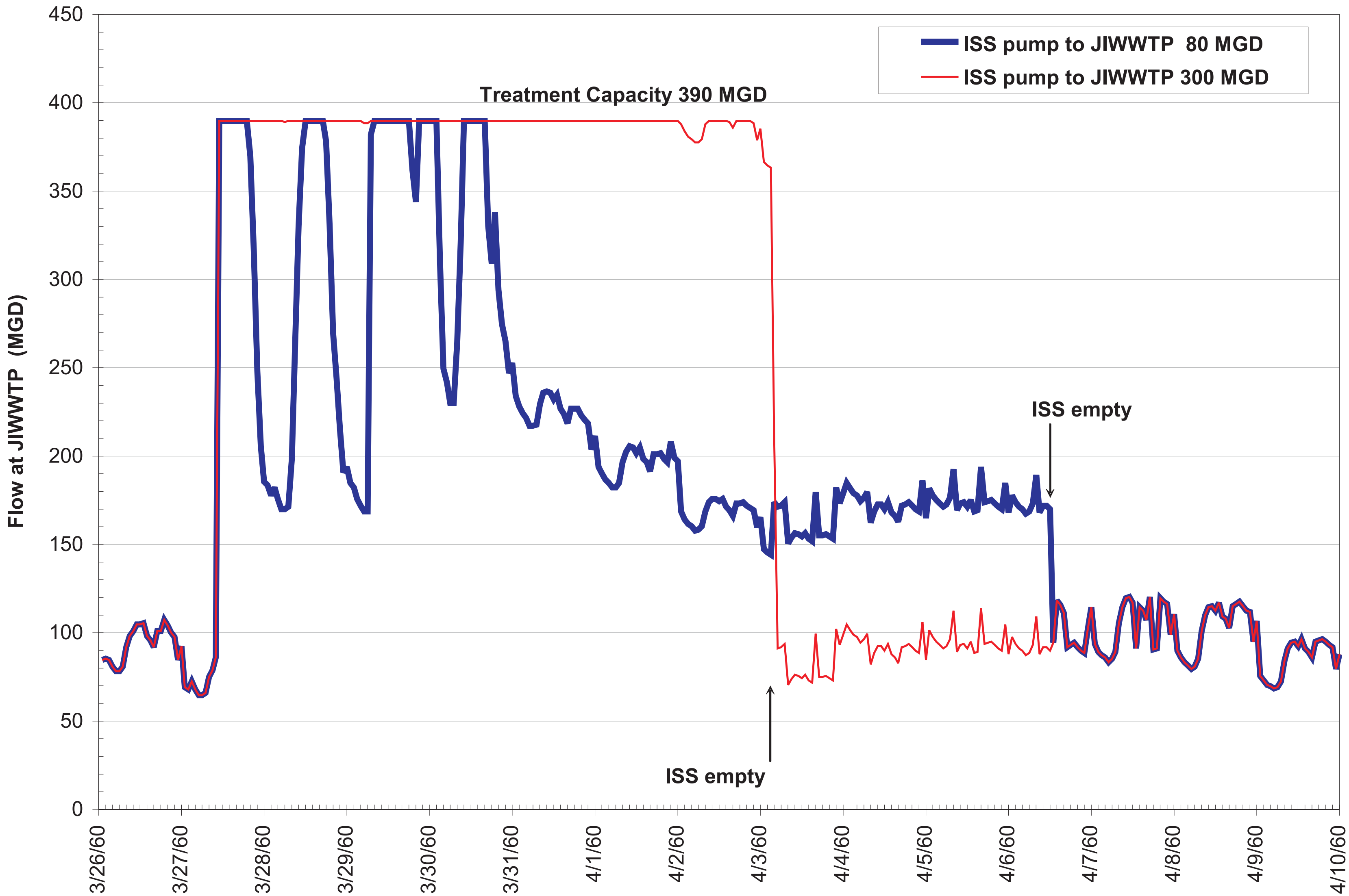

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FIGURE 3A-23
SIMULATED JIWWTP FLOW FOR MARCH 1960 AS A FUNCTION OF ADDITIONAL ISS PUMP CAPACITY
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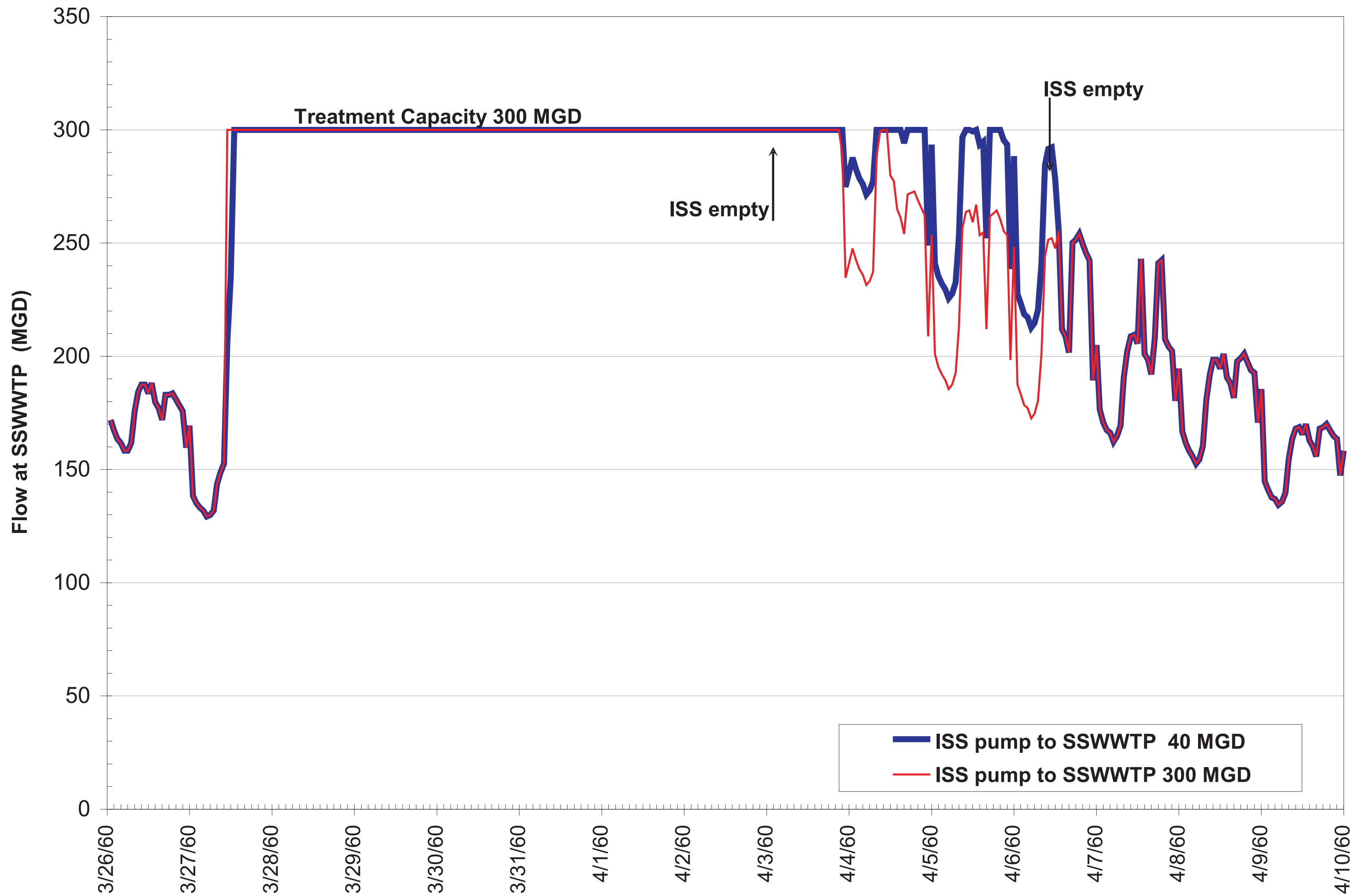

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FIGURE 3A-24

SIMULATED SSWWTP FLOW FOR MARCH 1960 AS A FUNCTION OF ADDITIONAL ISS PUMP CAPACITY

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Each hydrologic event has a unique timing and flow pattern. The March 1960 event is a large SSO event and is an example of the benefits of pumping to JIWWTP and the lack of opportunity to pump to SSWWTP. Additional pumping to SSWWTP can be a benefit for some hydrologic conditions. In general, the benefits of pumping to JIWWTP are greater than those for SSWWTP for most events.

The production functions, shown on figures 3A-25 through 3A-28, are based on the MACRO simulations using the full period of record. Consequently, the production functions account for the benefits of ISS pumping over a wide range of hydrologic conditions.

Production Functions – Inline Storage System Pumping to Jones Island Wastewater Treatment Plant

The production functions for volume removed by ISS pumping to JIWWTP are shown in Figure 3A-25. Additional ISS pumping to JIWWTP can remove both CSO and SSO volumes; the volume of CSO removed is similar to the volume of SSO removed. Figure 3A-26 shows the number of overflow events eliminated by ISS pumping to JIWWTP.

Production Functions – Inline Storage System Pumping to South Shore Wastewater Treatment Plant

The production functions for volume removed by ISS pumping to SSWWTP are shown in Figure 3A-27. Additional ISS pumping to SSWWTP can remove a small volume of CSO but not SSO. Figure 3A-28 shows the number of overflow events eliminated by ISS pumping to SSWWTP.

Cost Function

The unit cost for pumping to SSWWTP and JIWWTP are presented in Figure 3A-29 in the units of millions of dollars per million gallons per day treated ($M/MGD). The cost of the pumping to SSWWTP is approximately three times more than the cost to pump to JIWWTP because of the construction and O&M costs of the force main.

The cost functions for additional ISS pumping are shown in Figure 3A-30. Equations for cost as a function of additional pump capacity are also shown on the figure.

Cost Benefit Curves – Sanitary Sewer Overflow Indicator

The SSO and CSO cost benefit curves for pumping to SSWWTP and JIWWTP are presented in Figures 3A-31 through 3A-34. The slopes of the curves for pumping to JIWWTP are relatively flat which indicates good cost benefit relationships. The slope of the curves for pumping to SSWWTP are vertical for SSOs and steep for CSOs; this means it is not cost effective to pump to SSWWTP.

Summary: Inline Storage System Pumping

Pumping to JIWWTP is less expensive than pumping to SSWWTP because JIWWTP has treatment capacity after the peak of an event while there is volume in the tunnel. Pumping to SSWWTP is approximately three times more expensive due to the length of the force main required. Furthermore, because the flow to SSWWTP remains high for an extended period after the peak of the event, there is little opportunity to use additional pumping capacity to SSWWTP.

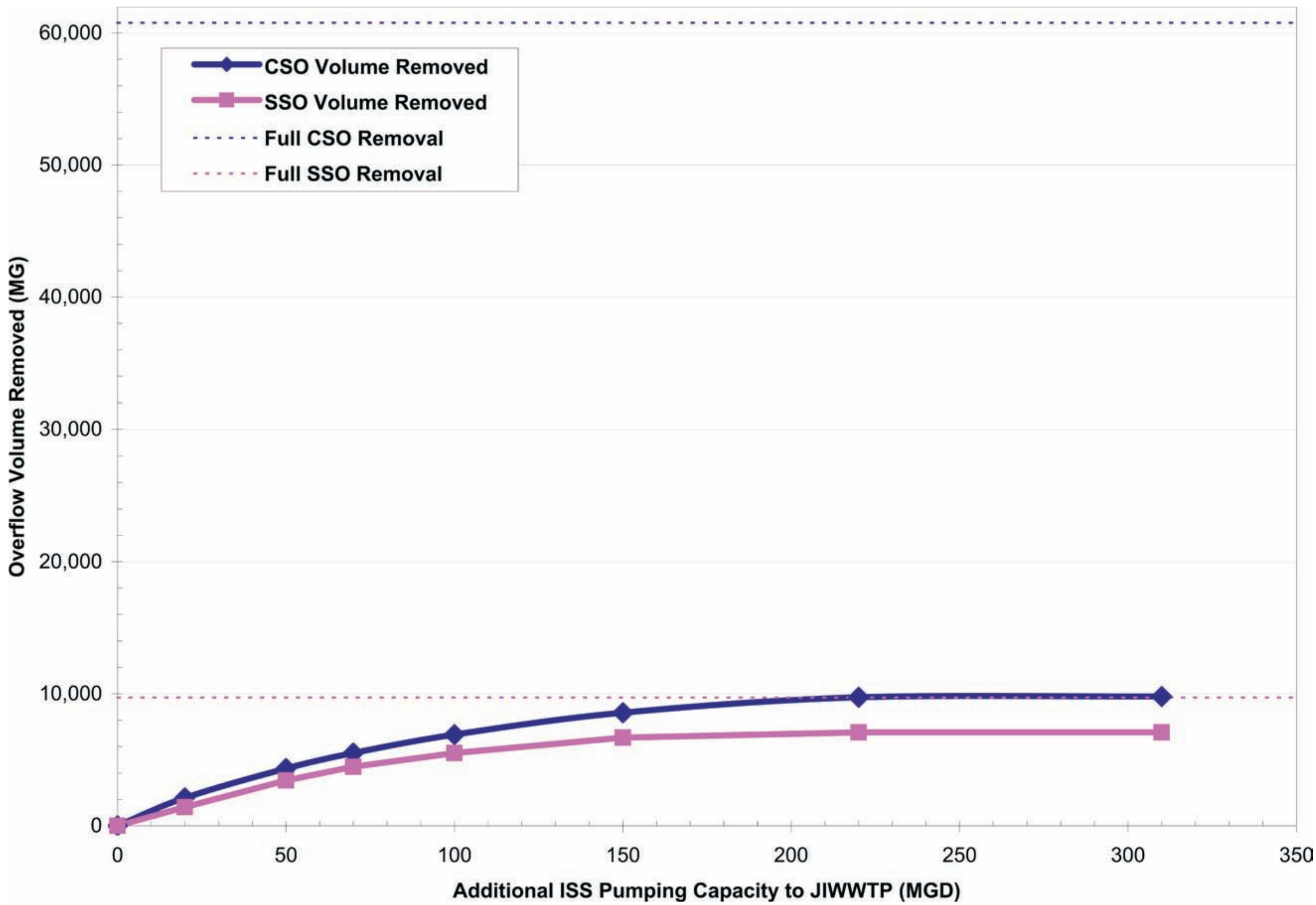

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FIGURE 3A-25

OVERFLOW VOLUME PRODUCTION FUNCTIONS: ADDITIONAL ISS PUMPING TO JIWWTP

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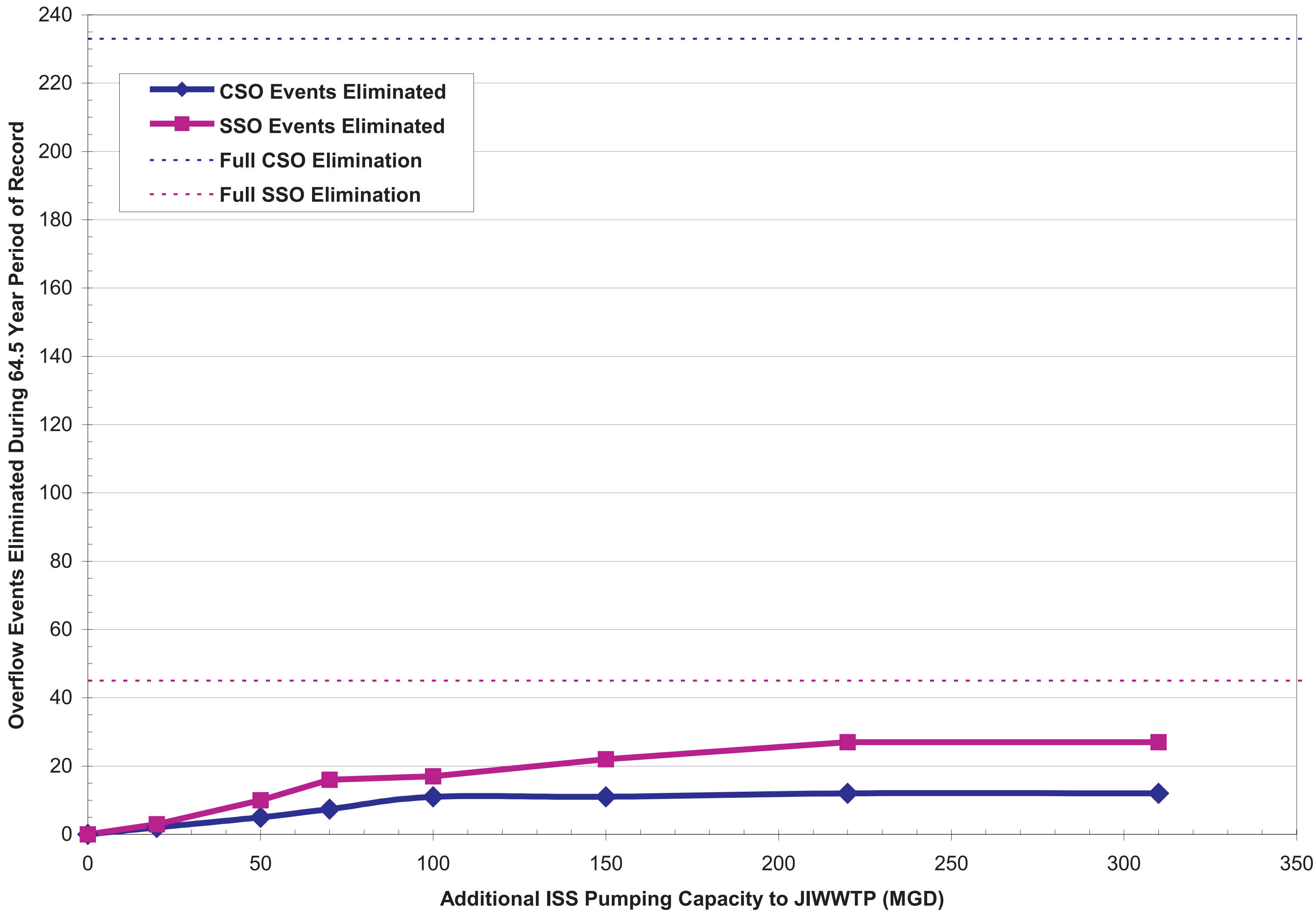

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FIGURE 3A-26

OVERFLOW EVENT PRODUCTION FUNCTIONS: ADDITIONAL ISS PUMPING TO JIWWTP

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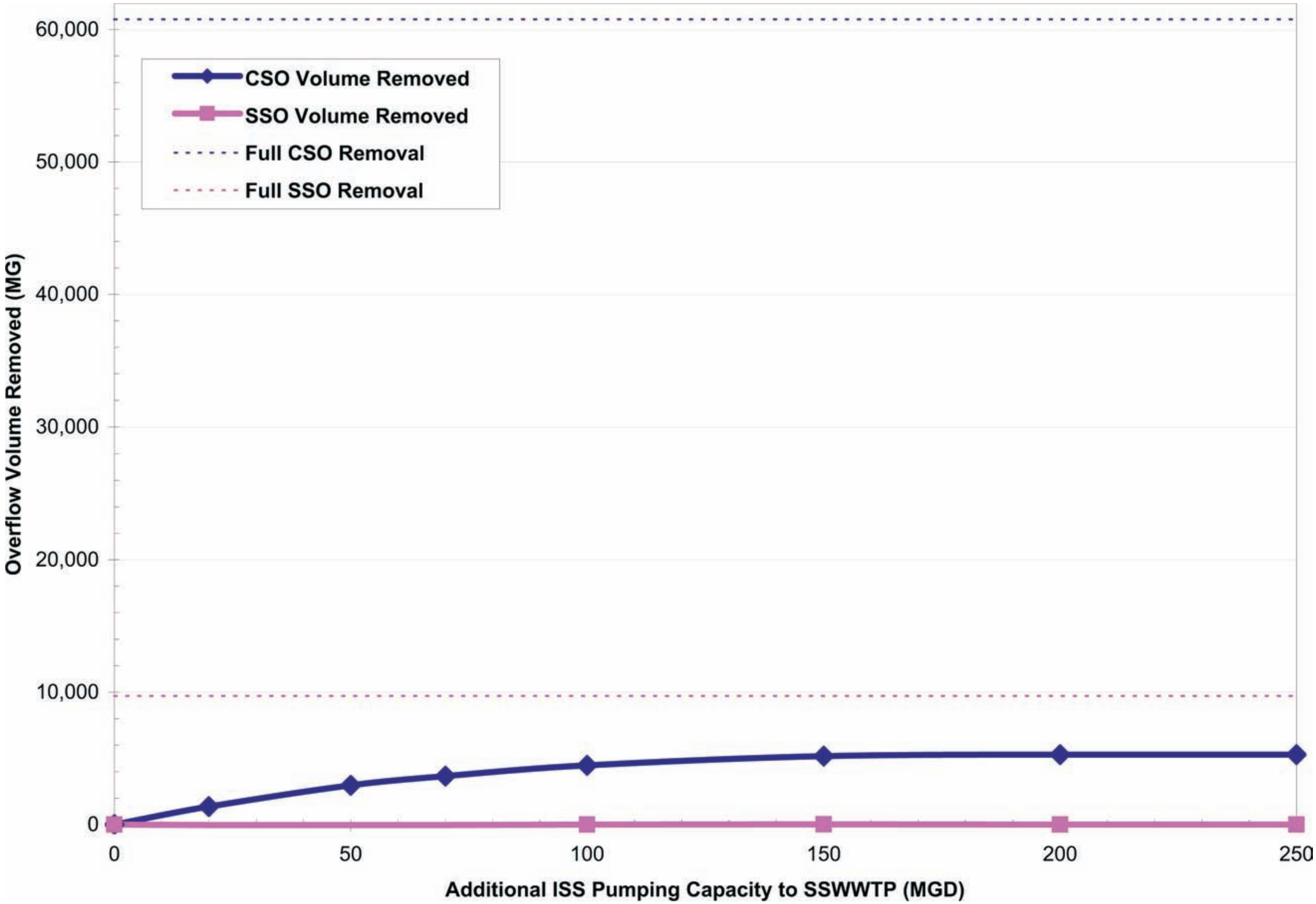

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FIGURE 3A-27
OVERFLOW VOLUME PRODUCTION FUNCTIONS: ADDITIONAL ISS PUMPING TO SSWWTP
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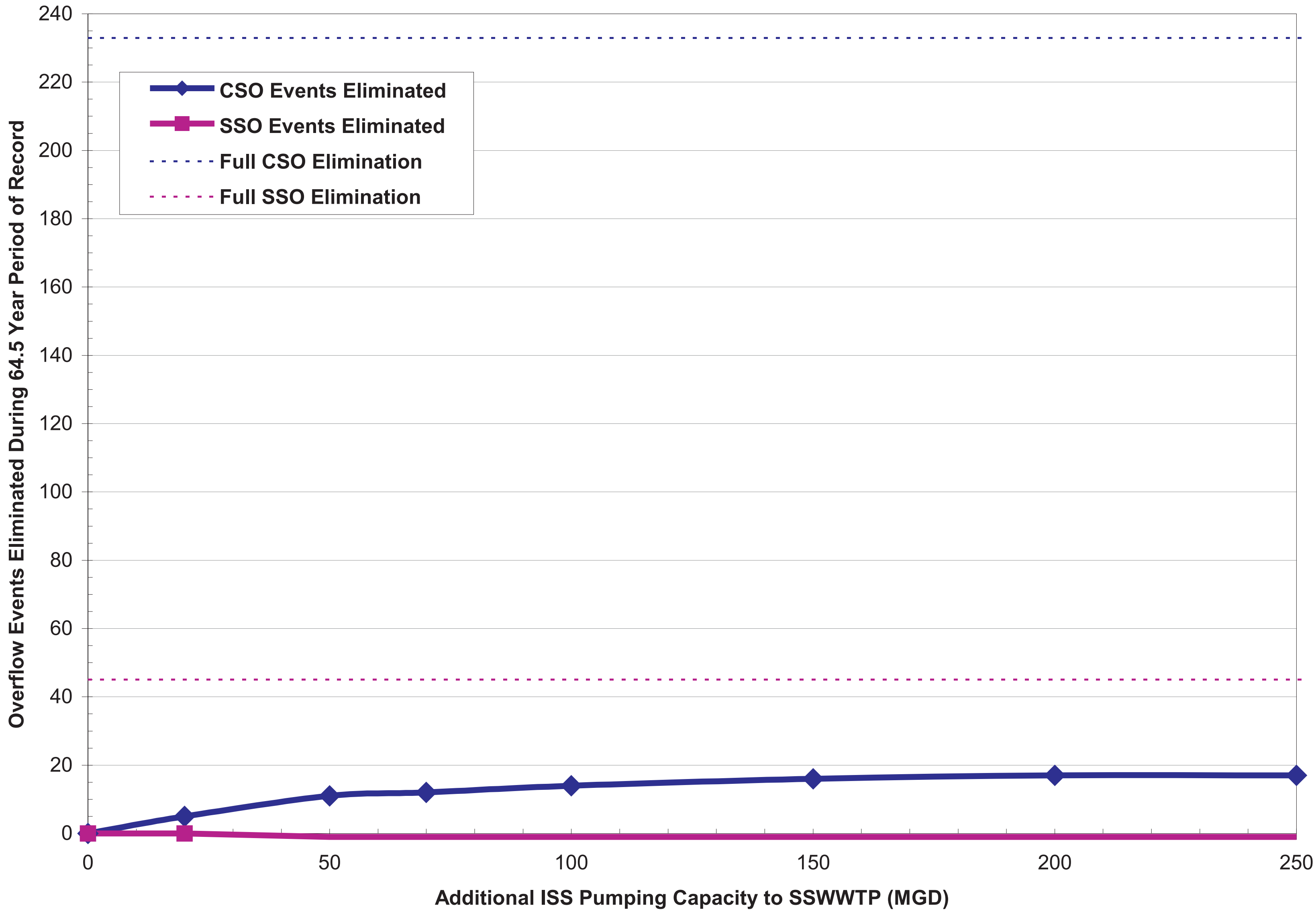

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FIGURE 3A-28

OVERFLOW EVENT PRODUCTION FUNCTIONS: ADDITIONAL ISS PUMPING TO SSWWTP

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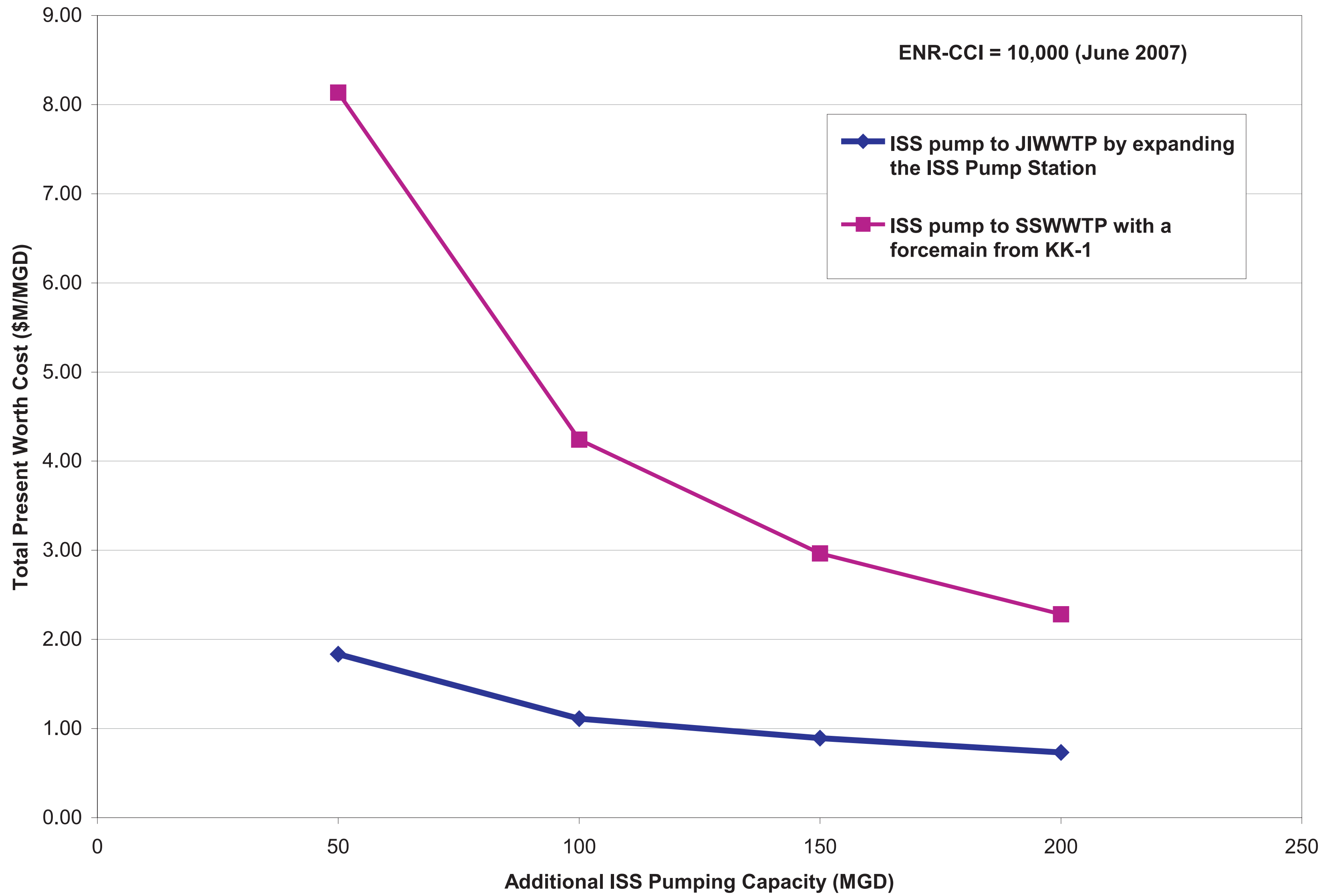

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FIGURE 3A-29
UNIT COSTS: ADDITIONAL ISS PUMPING
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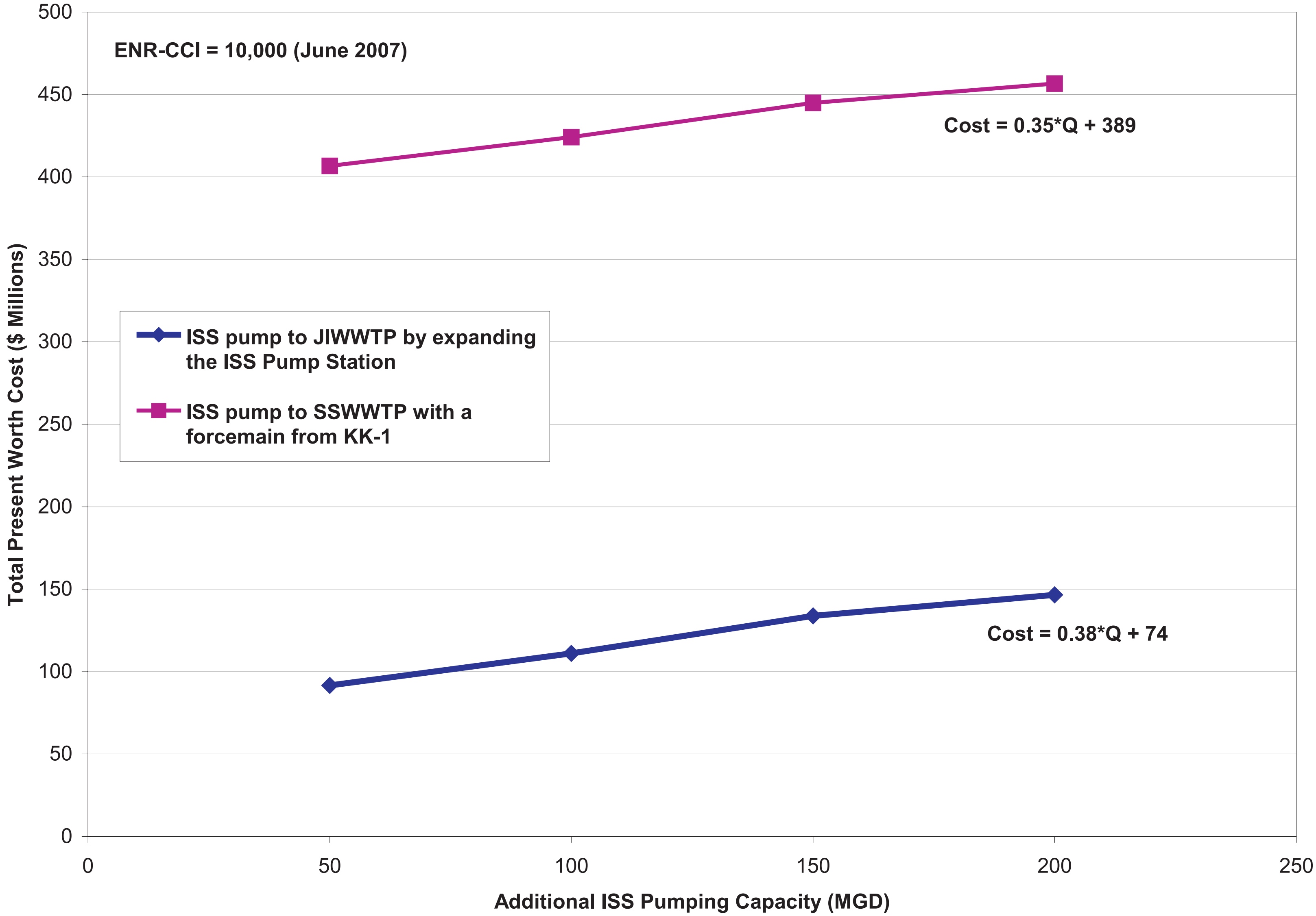

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FIGURE 3A-30

COST FUNCTIONS: ADDITIONAL ISS PUMPING

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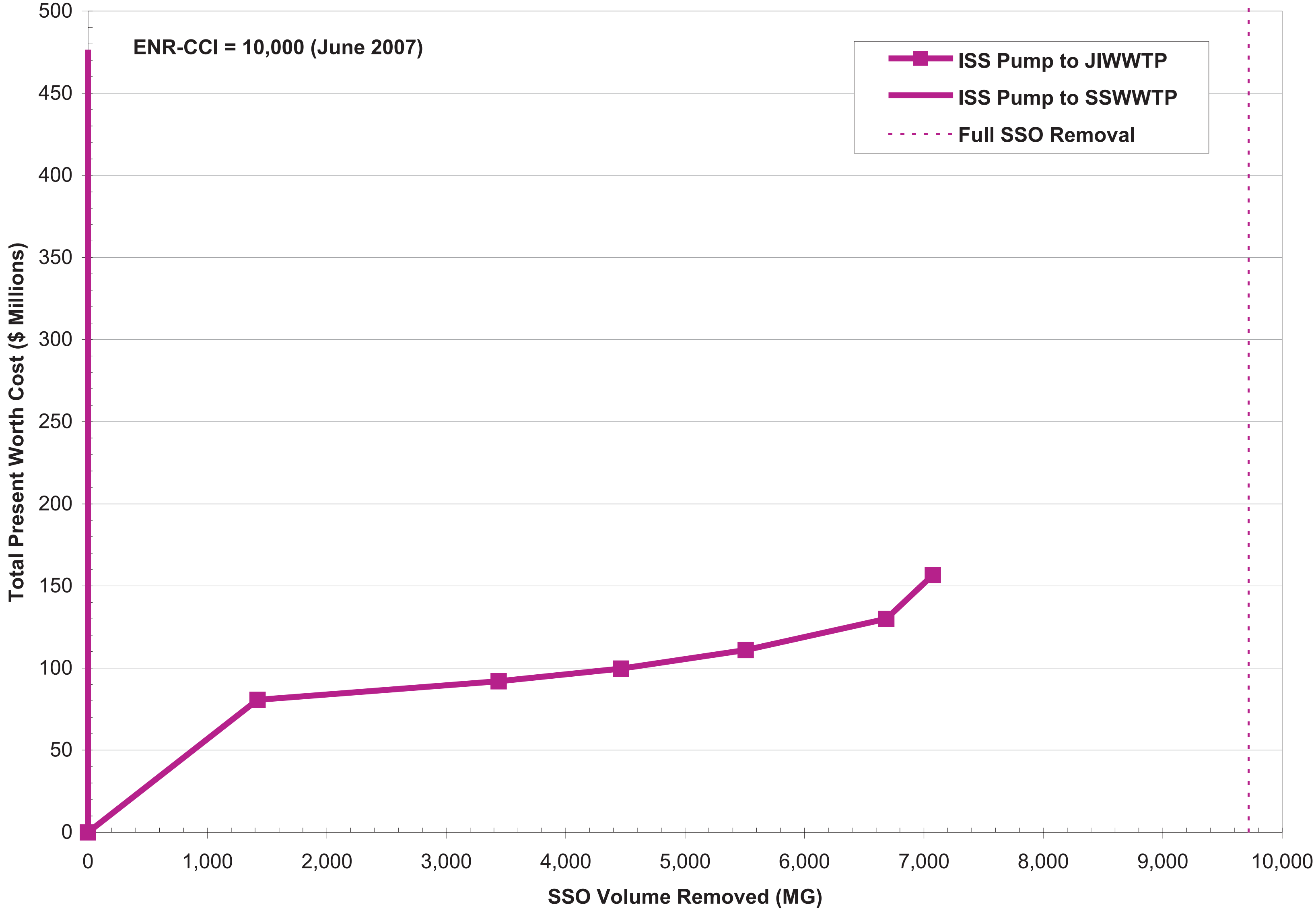

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FIGURE 3A-31
SSO VOLUME COST BENEFIT CURVES: ADDITIONAL ISS PUMPING
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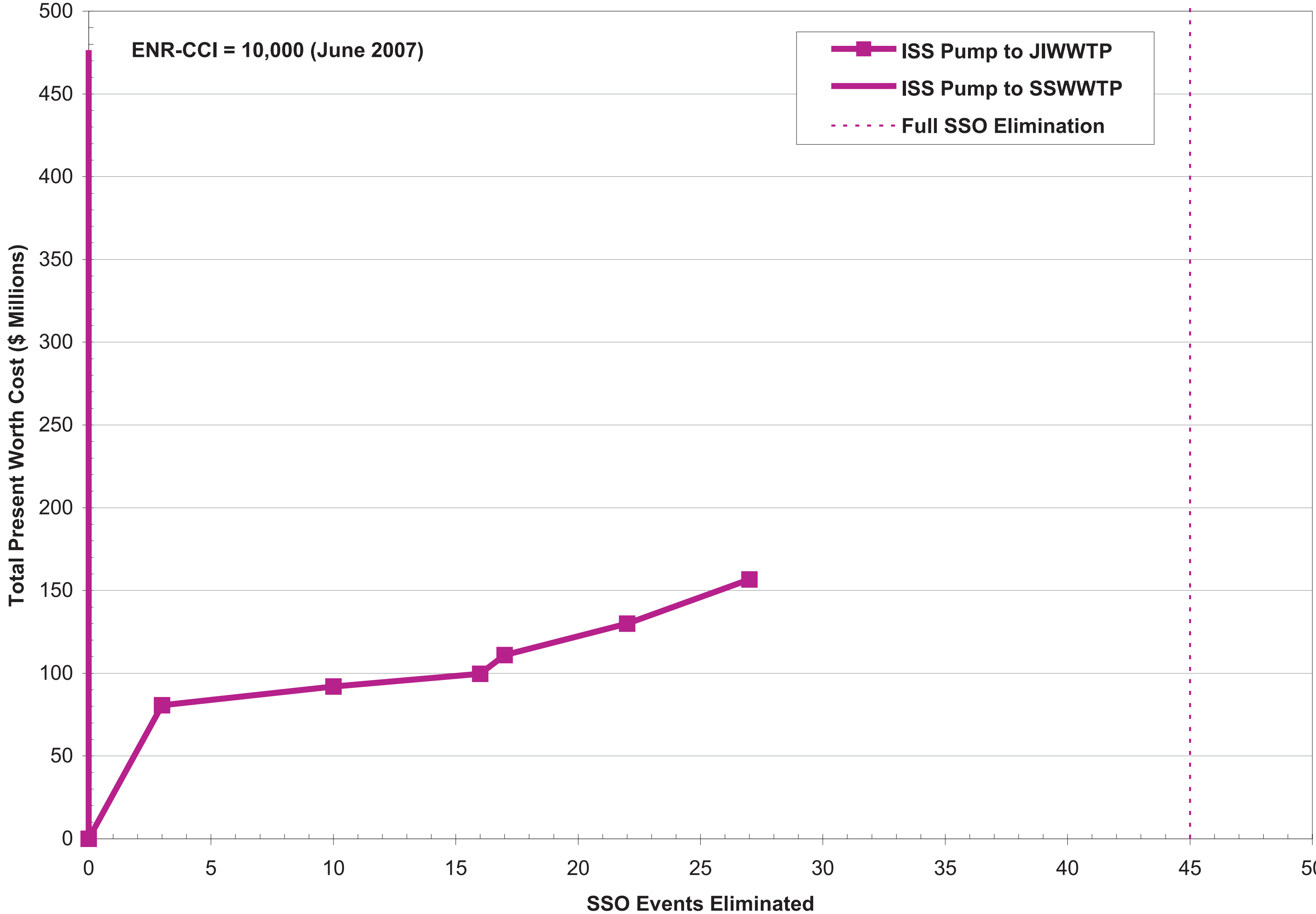

FIGURE 3A-32
SSO EVENT COST BENEFIT CURVES: ADDITIONAL ISS PUMPING
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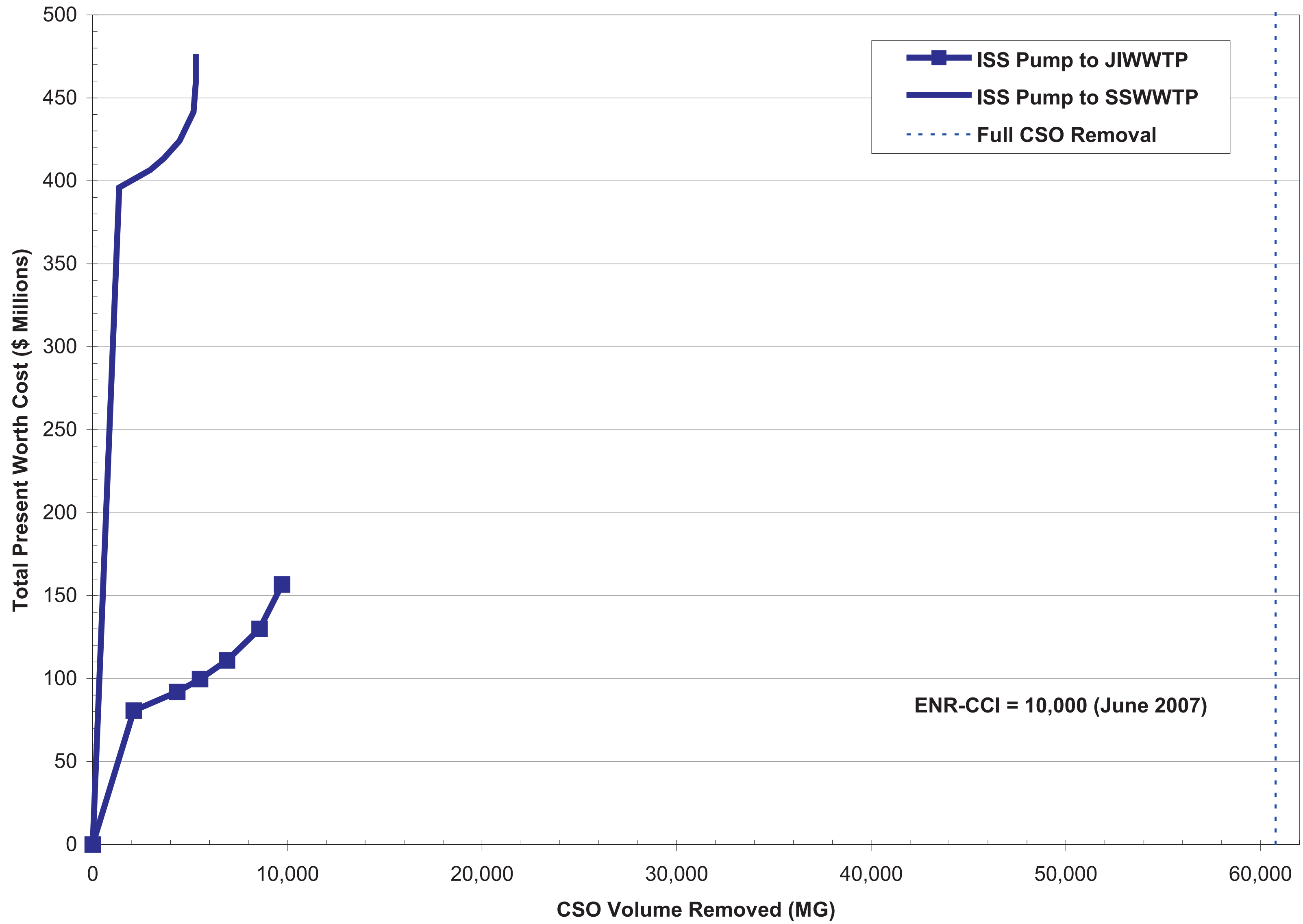

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FIGURE 3A-33

CSO VOLUME COST BENEFIT CURVES: ADDITIONAL ISS PUMPING

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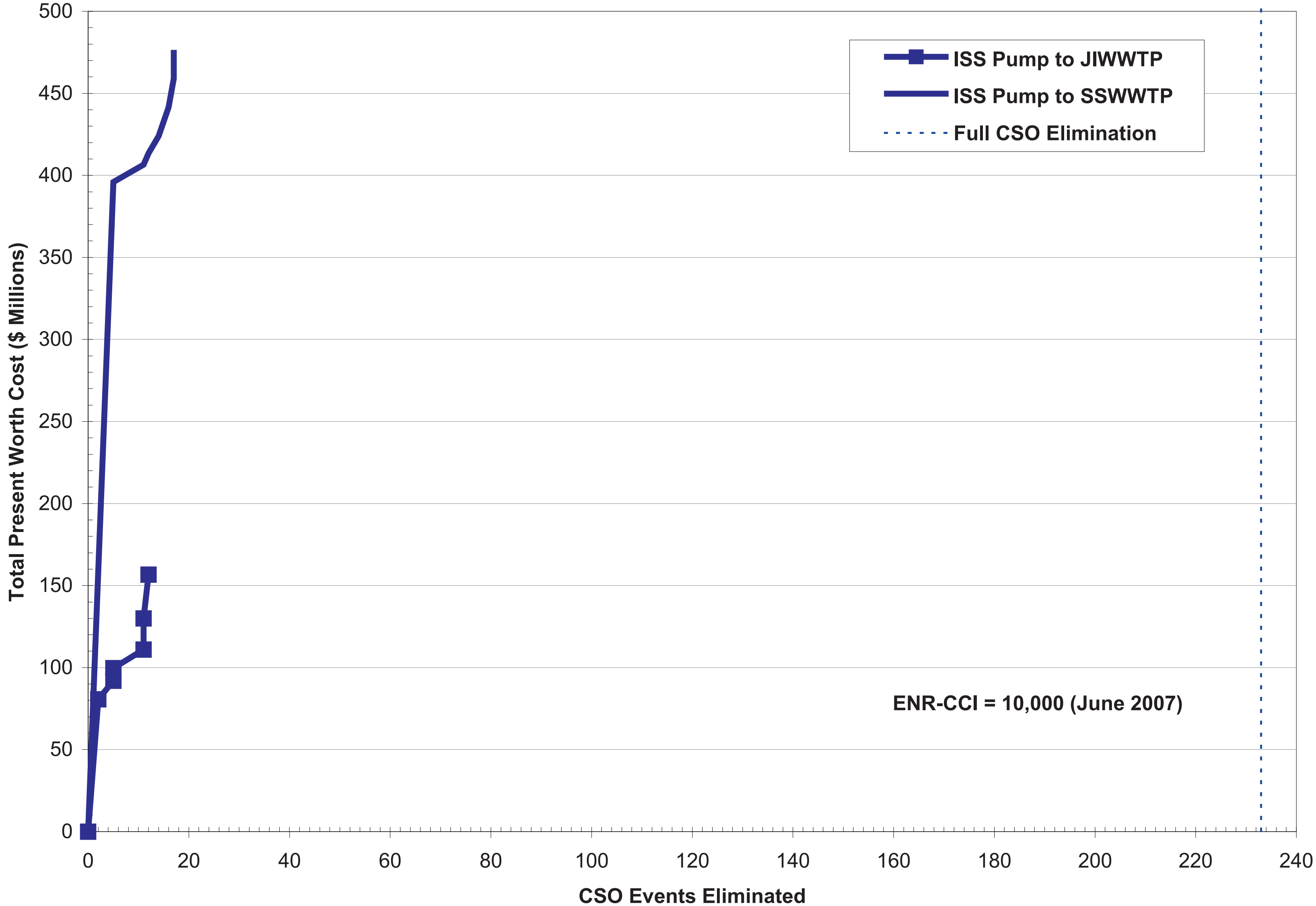

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FIGURE 3A-34

CSO EVENT COST BENEFIT CURVES: ADDITIONAL ISS PUMPING

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3A.2.5 Technology: Covered Near Surface Storage

Introduction

Covered near surface storage facilities can provide storage in portions of either the SSSA or the CSSA. Excess flow in a MIS adjacent to a storage facility is diverted into the near surface storage facility. Unlike the deep tunnel which provides storage for the entire system, near surface storage is dedicated storage for a specific part of the MIS system. Near surface storage can, however, have a broader impact by freeing up tunnel capacity for flow from other parts of the service area.

The production functions were generated using the MACRO model of the MMSD system. There is a set of production functions for near surface storage in the CSSA and another set of production functions for near surface storage in the SSSA.

Design Factors

For near surface storage facilities in the CSSA, the production functions extend up to a total volume of 2,000 MG. This near surface storage volume would not be in a single large facility. Rather, the total volume is the sum of many smaller facilities that have sizes based on site availability. For the purpose of this discussion, the typical facility size is assumed to be 25 MG; which is the typical size of the near surface storage alternatives considered for other MMSD projects such as the Port Washington Road Relief Sewer. Therefore, the largest total volume of 2,000 MG should be interpreted as 80 smaller facilities distributed around the CSSA, each with a typical size of 25 MG.

For the SSSA, the total volume for the largest case is 500 MG, which should be interpreted as 20 smaller facilities in the SSSA, each with a typical size of 25 MG.

Production Functions

The CSO and SSO production functions are shown in Figure 3A-35 for the volume of overflow removed by near surface storage. Near surface storage facilities in the CSSA can be very effective in reducing CSOs but not SSOs.

Figure 3A-36 show the number of overflow events eliminated by near surface storage. Storage in the CSSA is effective to reduce CSOs while storage in the SSSA is effective to reduce SSOs.

Cost Function: Covered Near Surface Storage

The unit cost of near surface storage facilities varies with site specific factors more than the size of the facility. The unit cost used for the cost function is based on estimates for the Port Washington Road study and actual construction costs of facilities in Racine and Muskego.

Other estimates were considered in the unit cost evaluation. One estimate is from a pump station in Elm Grove that could be equipped with a storage tank. Two other estimates are from proposed facilities in Minnesota.

In the Port Washington Road study, two near surface storage facilities were considered; the Green Tree Storage Facility with 22 MG of storage and the Humboldt Storage Facility with 24 MG.

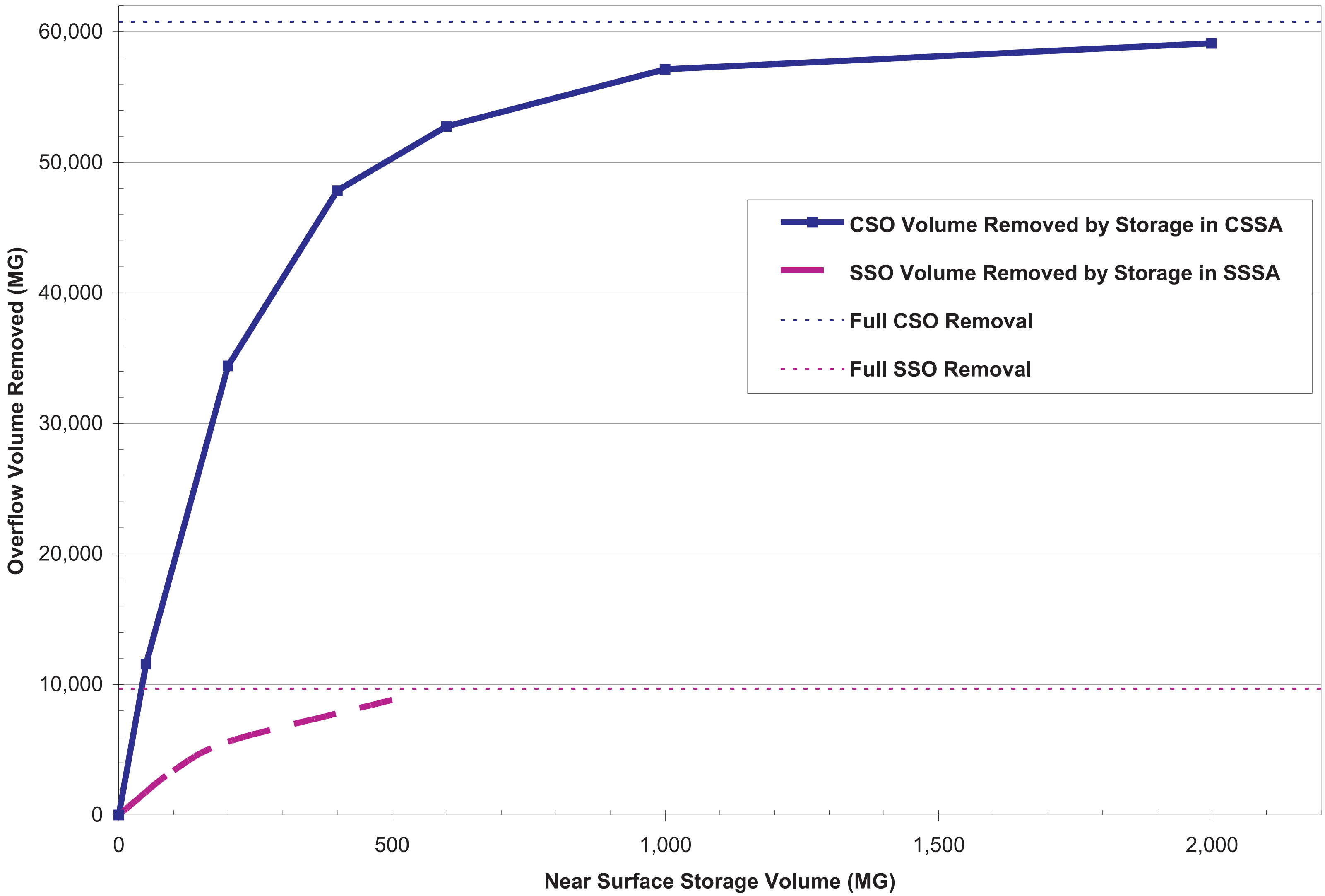

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FIGURE 3A-35
OVERFLOW VOLUME PRODUCTION FUNCTIONS: NEAR SURFACE STORAGE
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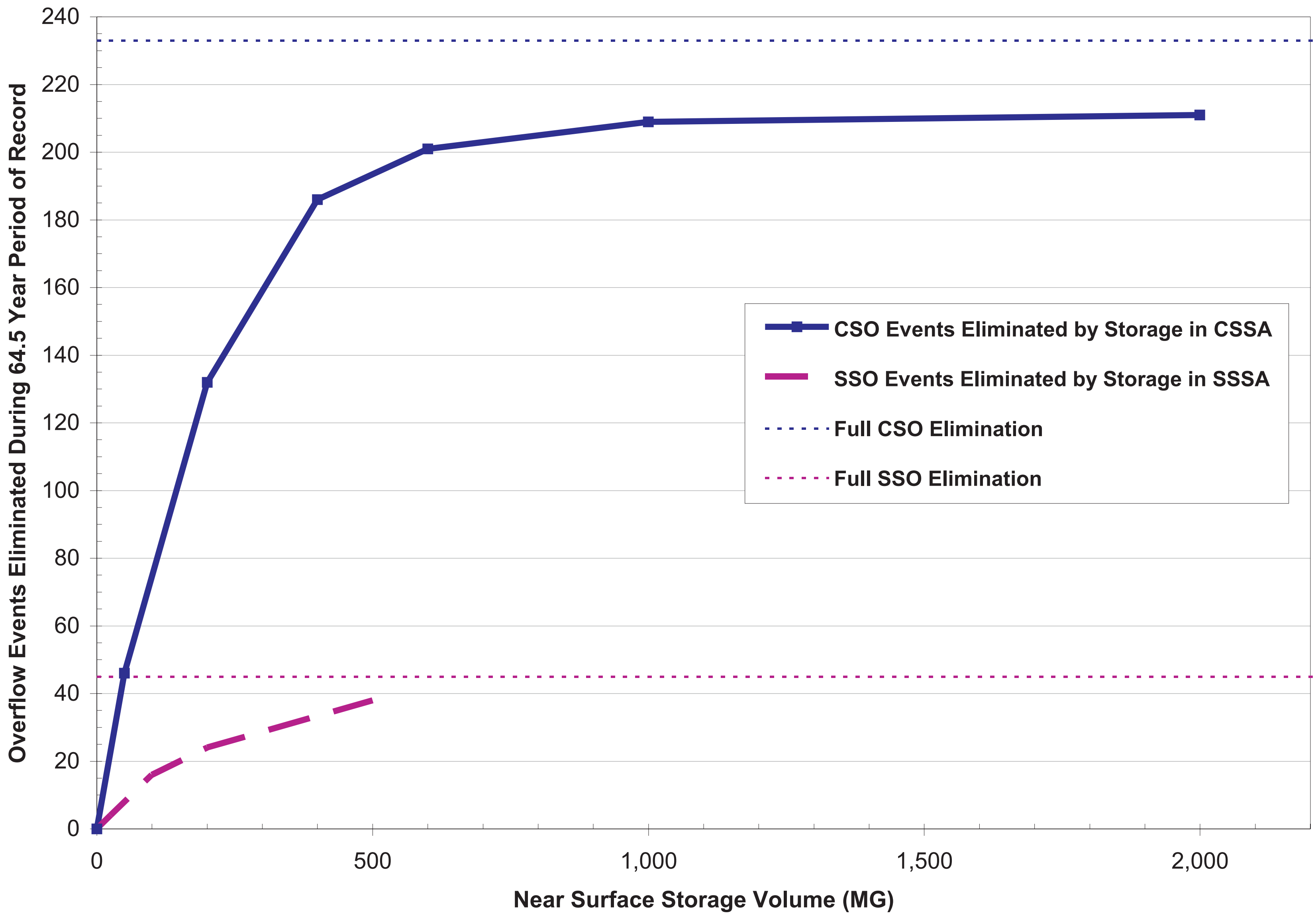

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FIGURE 3A-36
OVERFLOW EVENT PRODUCTION FUNCTIONS: NEAR SURFACE STORAGE
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The Racine Remote Storage Facility was constructed in 2003-2004 at Racine's John H. Batten Airport; it has 8.4 MG of storage and the construction cost was $6.9 million. The Muskego McShane Storage Facility was constructed in 2002-2003 under a neighborhood baseball sports complex; it has 1.7 MG of storage and the construction cost was $3.25 million.

A proposed 0.8 MG facility in Elm Grove adjacent to a pump station at Squires Grove and Wrayburn Road was also used as a cost estimate along with two proposed facilities in Minnesota with 1.3 and 6.0 MG storage volumes.

These unit costs are shown in Figure 3A-37. The typical unit cost for facilities over 25 MG is $3.50/gallon.

The cost function uses a constant unit cost of $3.50/gallon. Because the production functions cover a range up to 2,000 MG, the cost function also uses the same range. The cost function is shown in Figure 3A-38. Assuming a typical size of 25 MG for each facility, the number of storage facilities needed to provide the total storage is also annotated on the cost function graph.

Cost Benefit Curves: Sanitary Sewer Overflow Indicator

The cost benefit curves show that SSOs are removed effectively by near surface storage facilities located in the SSSA; this is shown in Figure 3A-39 for SSO volume removed and Figure 3A-40 for SSO events eliminated.

Cost Benefit Curves: Combined Sewer Overflow Indicator

As shown in Figures 3A-41 and 3A-42, CSOs are effectively removed by facilities in the CSSA.

Summary: Covered Near Surface Storage

Benefits from covered near surface storage are limited, for the most part, to the MIS in the vicinity of the storage facility. If the storage facility is in the CSSA, there are direct benefits to reduce CSOs. Likewise, if the storage facility is in the SSSA, there is a direct benefit to SSOs in the vicinity of the facility.

The number of the storage facilities required to achieve a significant reduction in overflows is a significant drawback of this technology. The typical size for individual facilities is assumed to be 25 MG in this study. To eliminate most of the SSOs, over 20 sites in the SSSA would be needed. To eliminate most of the CSOs, over 80 sites in the CSSA would be needed. The difficulty of locating acceptable sites for near surface storage, particularly in the CSSA, is a major limitation for this technology.

3A.2.6 Technology: Metropolitan Interceptor Sewer In-System Storage

During wet weather, excess flow enters the deep tunnel system as the MIS reaches its capacity. Therefore, additional storage in the MIS will capture more of the wet weather volume before it enters the deep tunnel and those flows will not, therefore, require pumping to reach the WWTPs. While storage in this form is very efficient for SSO reduction, the amount of storage that can be added to the MIS is limited. It is likely that additional storage would be in the form of relief sewers that are larger than what is needed for conveyance relief, which would provide more volume than inflatable dams as was originally recommended for the technology identified in Chapter 2, *Technology/Indicator Analysis*. This analysis is not specific to any one relief site, but considers the cumulative benefit of many oversized relief sewers.

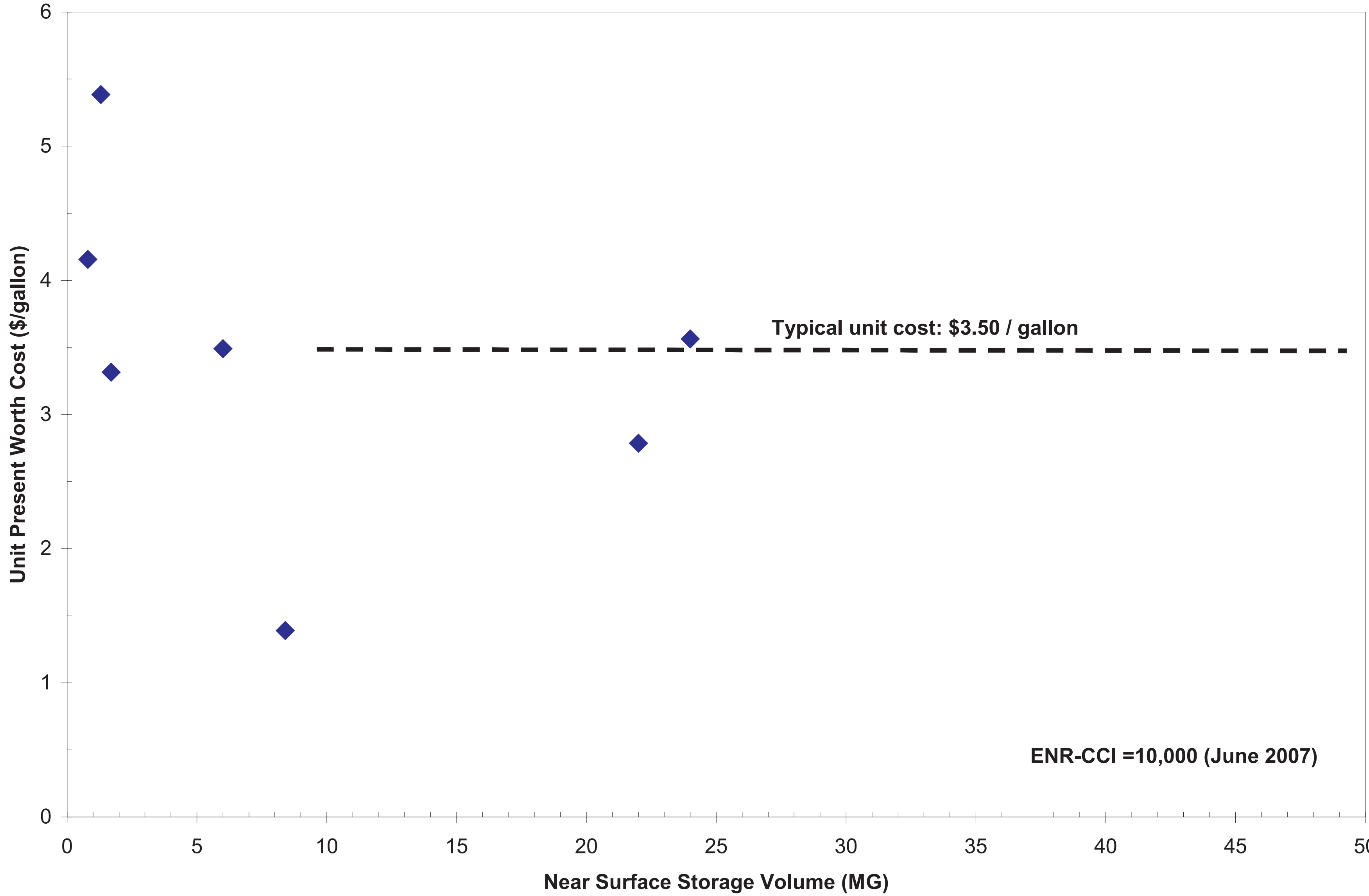

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FIGURE 3A-37
UNIT COSTS: NEAR SURFACE STORAGE FACILITIES
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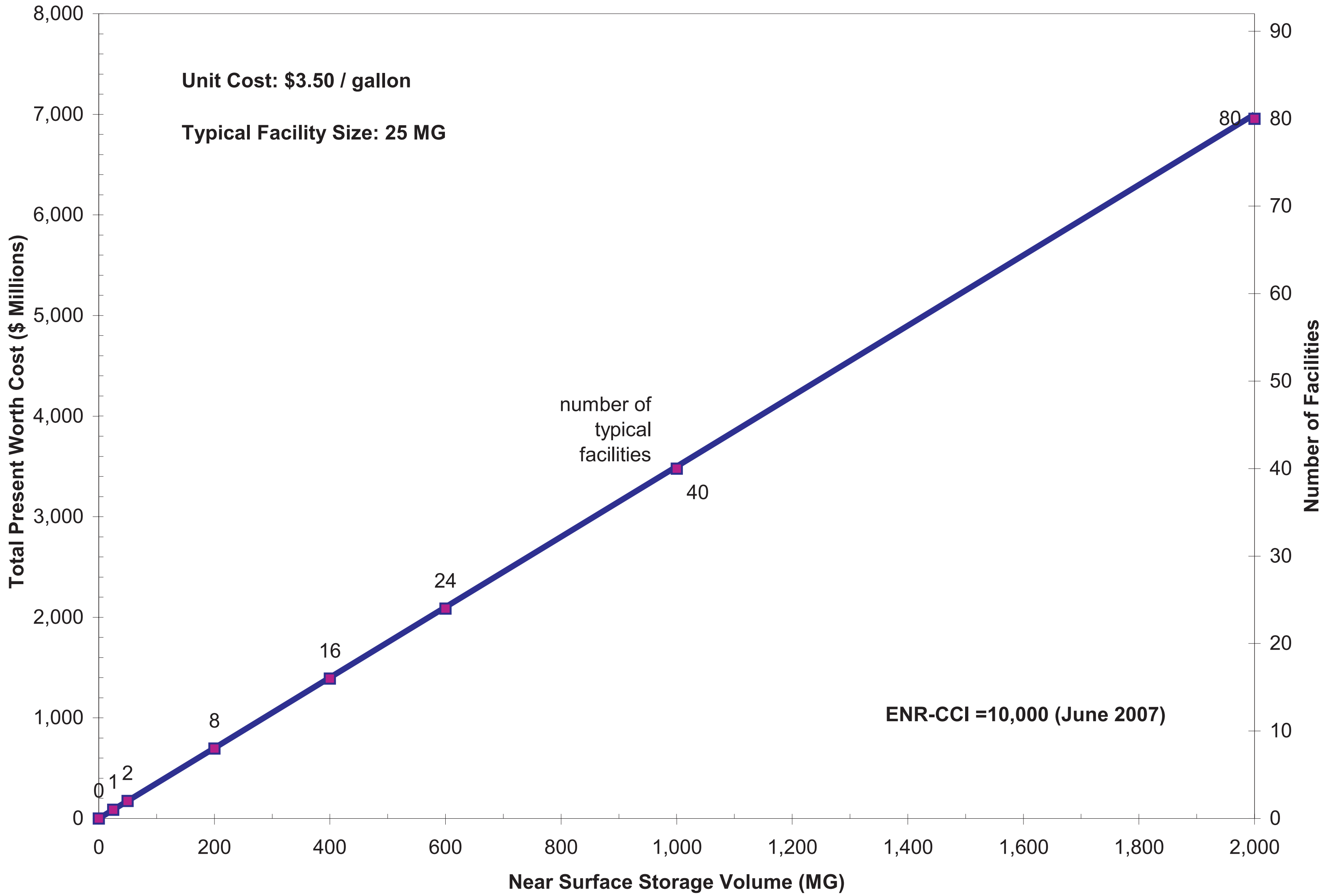

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FIGURE 3A-38
COST FUNCTION:
NEAR SURFACE STORAGE
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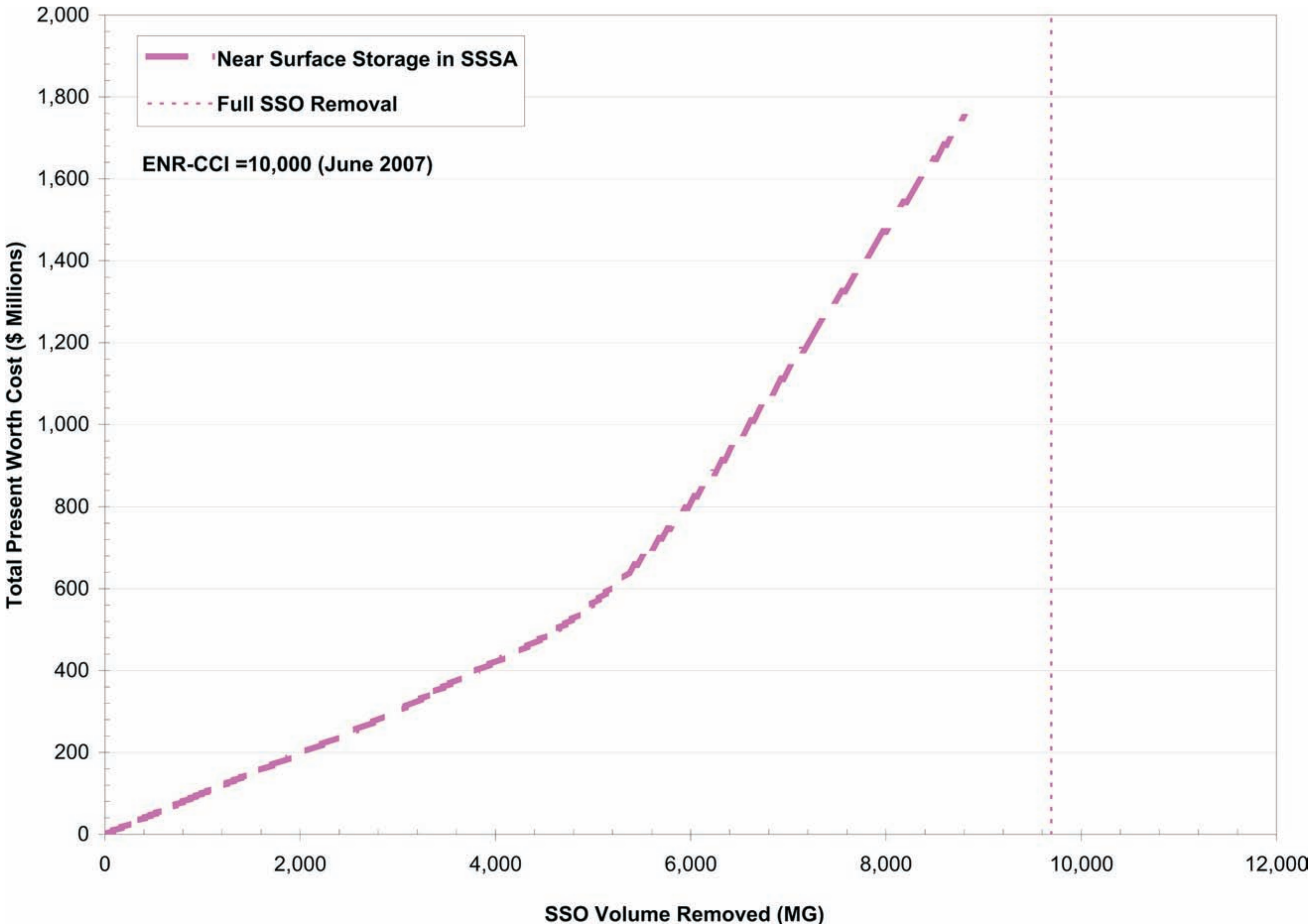

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FIGURE 3A-39
SSO VOLUME COST BENEFIT CURVE: NEAR SURFACE STORAGE
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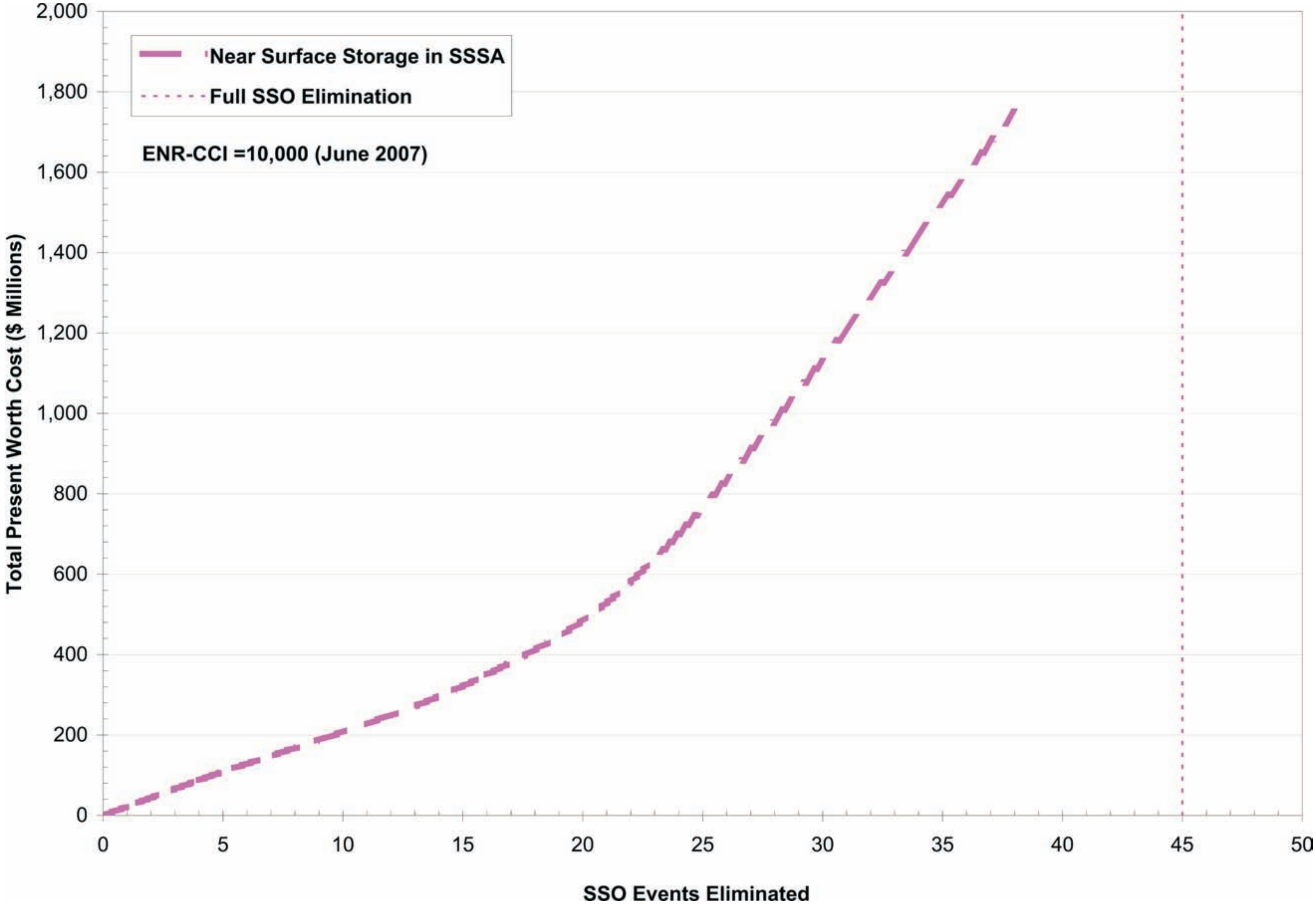

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FIGURE 3A-40

SSO EVENT COST BENEFIT CURVE: NEAR SURFACE STORAGE

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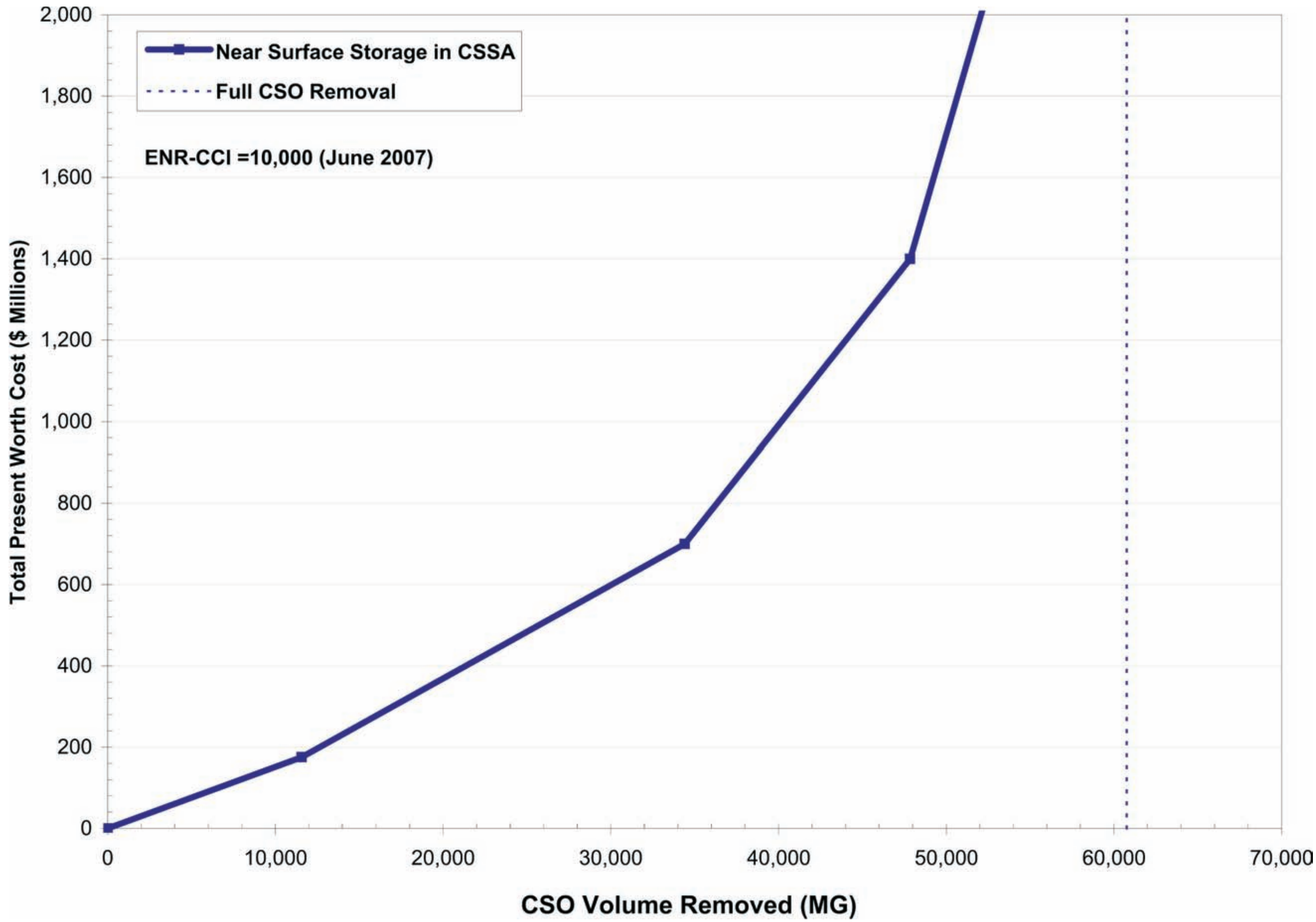

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FIGURE 3A-41
CSO VOLUME COST BENEFIT CURVE: NEAR SURFACE STORAGE
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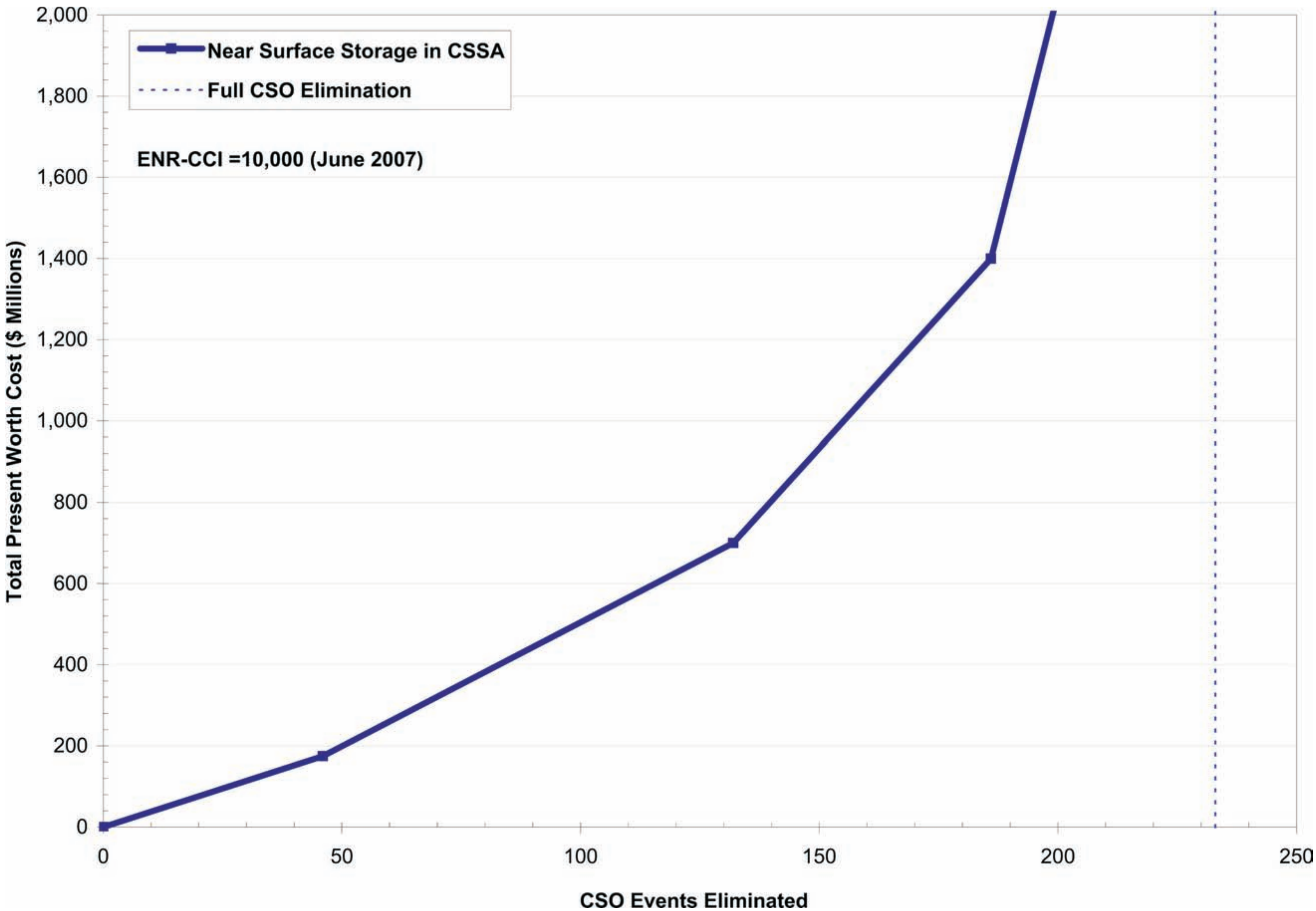

FIGURE 3A-42
CSO EVENT COST BENEFIT CURVE: NEAR SURFACE STORAGE
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Additional volume in the MIS could be achieved by oversizing new relief sewers. It is assumed that most relief sewers will be built in the MIS Subsystems 1-4 because these are the undeveloped areas that have the greatest potential for growth. Therefore, the MACRO subarea for subsystems 1-4 was assigned additional volume in this analysis.

Two simulations were run, one with 30 MG and another with 80 MG of additional volume in MIS Subsystems 1-4. No specific relief sewer was proposed in this analysis. These volumes would likely be the sum of several relief sewer projects. The modeling results demonstrate the overall sensitivity of SSO and CSO discharge to additional MIS volume rather than the detailed benefit of a site-specific project.

For example, if a 4-foot relief sewer is needed to provide relief from a bottleneck, additional storage in the MIS could be achieved by installing an oversized 10-foot pipe. The additional volume in the MIS available in a 10-foot pipe instead of a 4-foot pipe represents an additional storage volume of 2.6 MG/mile. This is the incremental volume of the 10-foot pipe above the full volume in the 4-foot pipe. In this example, an additional volume of 80 MG could be interpreted as approximately 30 miles of oversized relief sewers.

Production Function

The CSO and SSO reduction production functions are shown in Figure 3A-43; the scale of this figure is intentionally large so that it can be compared to other production function curves on the same scale. With this scale, it is clear that the extent to which MIS storage can be applied is relatively small compared to other technologies. The SSOs are reduced by a modest amount. The reduction in CSOs is negligible because in this analysis no additional volume was added to the MIS in the CSSA. Figure 3A-44 is the same image with a smaller “zoom-in” scale; in this figure the curves are at a scale that provides better insight into the relationship of CSO and SSO reduction production functions.

Cost Function: Additional Metropolitan Interceptor Sewer Storage

The cost function for additional MIS in-system storage represents the incremental cost for the oversized pipe above the base cost of a minimum sized sewer that would provide conveyance relief alone. The cost of additional MIS in-system storage should also include the cost of gates or other control structures required to take advantage of the excess volume and to prevent downstream problems from the extra conveyance capacity of the oversized pipe. To define the incremental cost, a specific case is needed to compare the cost of the oversized pipe to the minimum required pipe.

The O&M cost should also reflect the incremental cost of the oversized sewer relative to a smaller pipe and the cost of the control structure. All of these considerations are dependent on the conditions of a specific site; it is not possible to generate a cost function with this level of detail. Furthermore, due to the limited range of application, and the relatively small magnitude of the production functions, additional effort to generate a detailed cost function is not merited.

A simplified cost function can be created for the incremental construction cost of an oversized sewer using the typical construction cost values for various sized pipes.

Figure 3A-45 shows typical construction costs for wastewater pipelines. These values are based on the replacement cost of gravity sewers in residential areas.

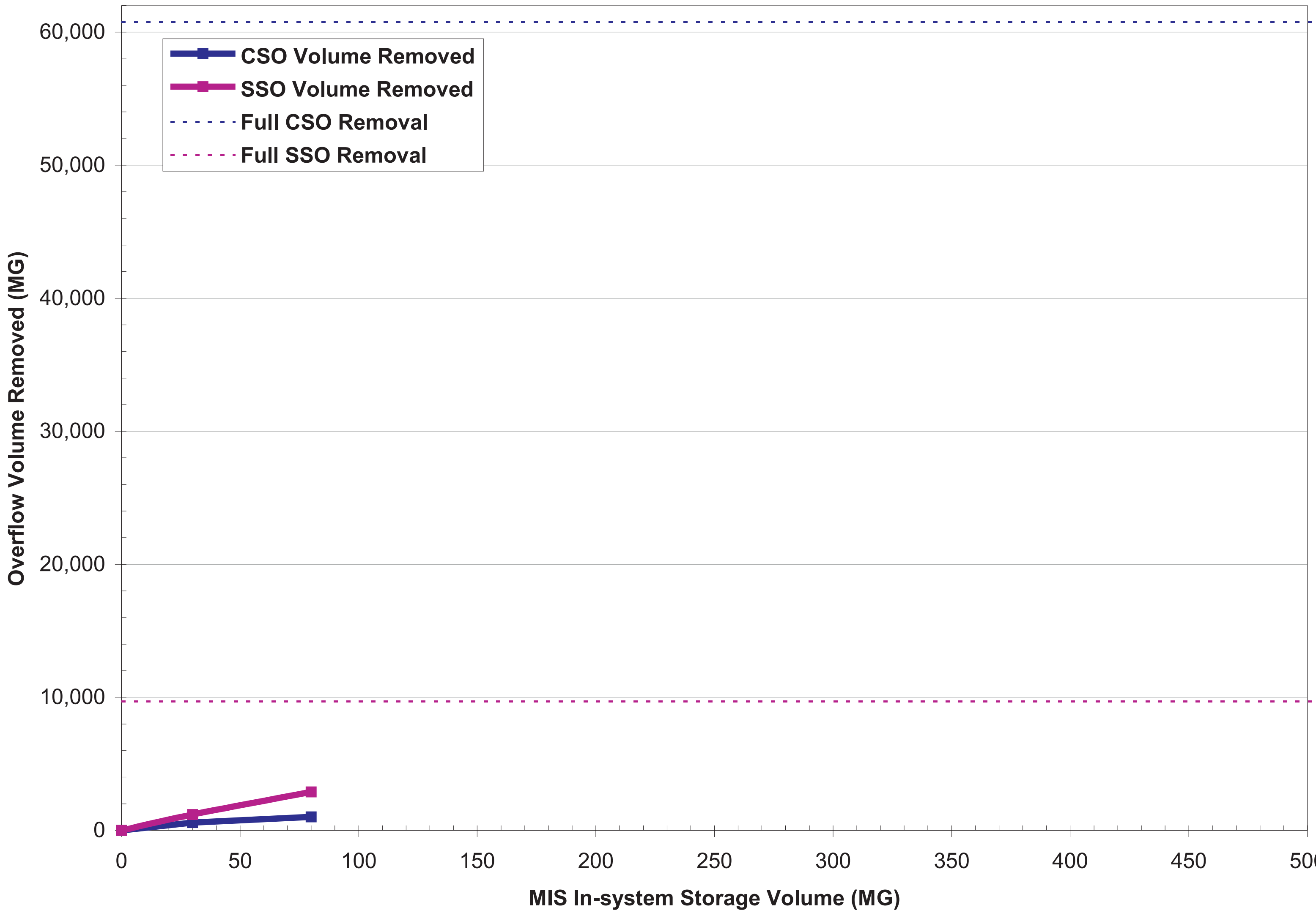

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FIGURE 3A-43
OVERFLOW VOLUME PRODUCTION FUNCTIONS: MIS IN-SYSTEM STORAGE
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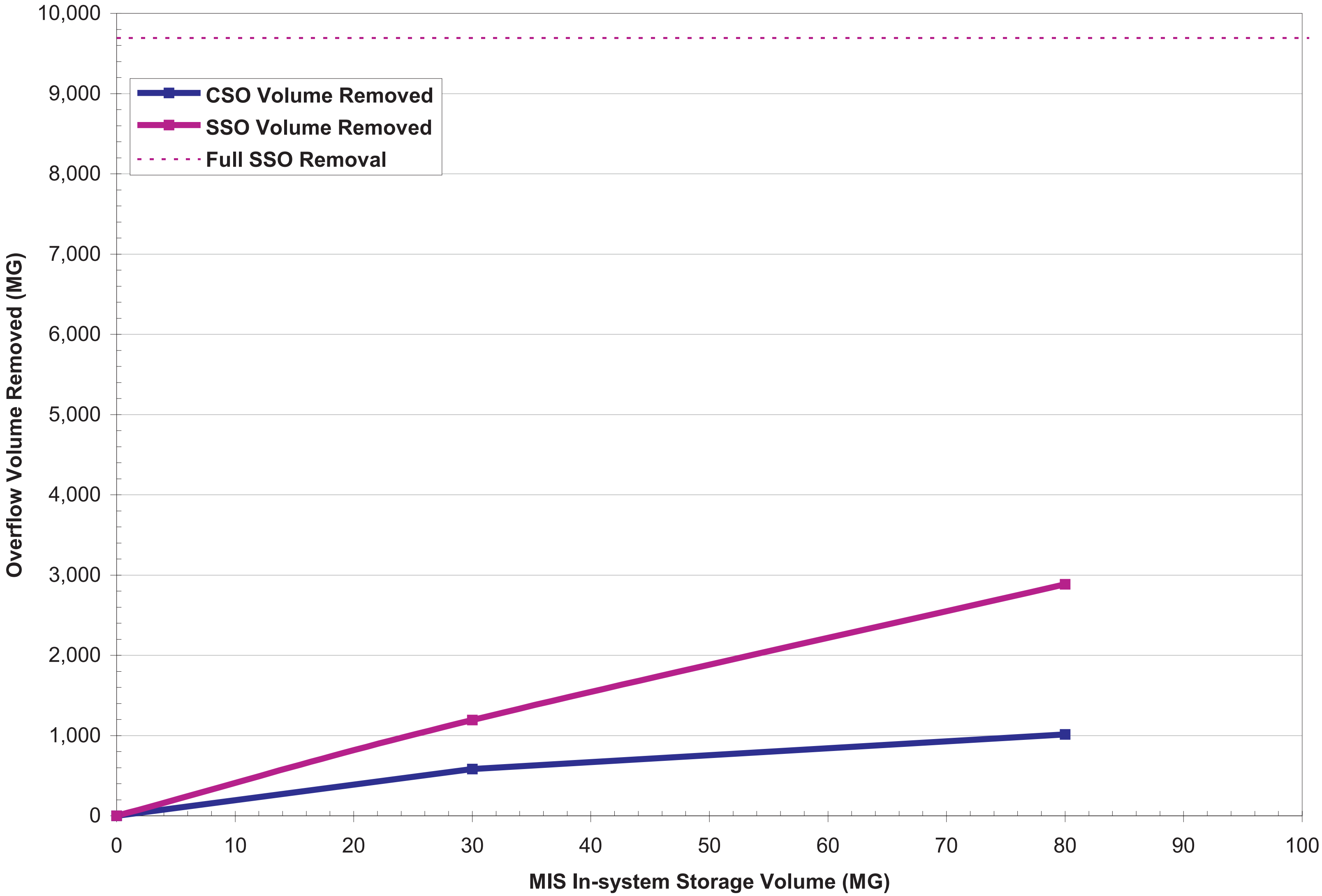

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FIGURE 3A-44

OVERFLOW VOLUME PRODUCTION FUNCTIONS: MIS IN-SYSTEM STORAGE (ZOOM-IN SCALE)

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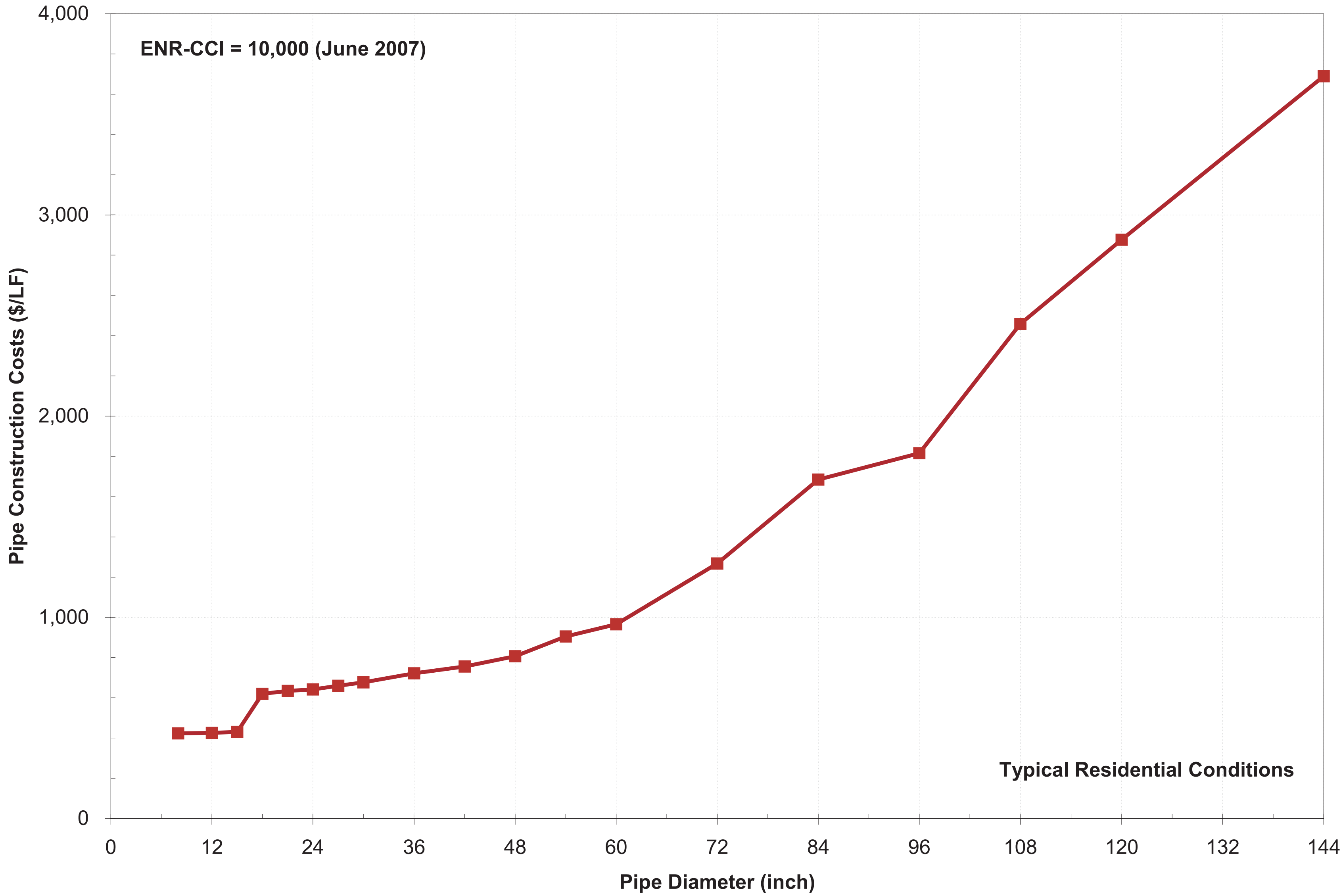

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FIGURE 3A-45

TYPICAL CONSTRUCTION COST FOR WASTEWATER PIPELINES

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A cost function for 10-foot oversized pipe in place of 4-foot pipe is presented in Figure 3A-46. The oversized pipe provides 2.6 MG/mile of in-system storage. The cost of the 10-foot pipe is $2,900/ft and the 4-foot pipe is $800/ft. After accounting for contingencies of 25% and non-construction cost of 25%, the incremental unit cost of the oversize pipe is $6.56/gallon.

The cost benefit curves for SSO are shown in Figures 3A-47 and 3A-48.

Summary: Metropolitan Interceptor Sewer In-System Storage

Additional storage in the MIS could be provided by oversizing relief sewers. The storage created by the oversized sewers may provide benefits in the vicinity of the relief sewer but the system-wide benefit is small.

3A.2.7 Technology: Inlet Restrictors

Inlet restrictors in the CSSA can temporarily detain stormwater in the roadways to reduce the peak flow rate in the combined sewer system. The volume of stormwater that can be stored in this way is limited; consequently, the CSO reduction due to inlet restrictors is small.

The inlet restrictor production functions for CSO and SSO reduction volumes are based on the MACRO model results for near surface storage in the CSSA. Only the cost function is distinct for inlet restrictor technology.

Potential Areas to be Served by Inlet Restrictors and Depth of Ponding

Inlet restrictors detain stormwater in the curb and gutter areas of local streets in the CSSA. Land use information from SEWRPC was used to define the area for potential application of inlet restrictors. The residential part of the CSSA is 8,600 acres. Only roadway/parking land use areas associated with recreational, residential, or local-collector streets were used. Roadways were not considered if they were associated with arterial streets, expressways, freeways, commercial, industrial, or governmental institutions. Roadways that qualified as candidates for inlet restrictors have a total area of 2,757 acres, which is 17% of the CSSA.

Due to street slope and the allowable maximum depth of ponding, only a fraction of a street is able to store water. It is assumed that up to 10% of the possible roadway area could be ponded; this depends on topography, curb depth, and the berm design. It is unlikely that all of this area would actually be available for ponding when other constraints are satisfied. Therefore, this analysis represents an upper bound assuming the fullest implementation of the technology.

It is undesirable for ponding to spread beyond the limits of the roadway. The maximum depth can be 6 to 8 inches and still safely allow vehicle traffic, but due to the camber of the road and the wedge shaped volume of the ponded water on a slope, the average depth of ponding is assumed to be 4 inches. Based on the above assumptions the maximum volume of stormwater that could be detained by inlet restrictors is 30 MG.

To evaluate the impact of detaining 30 MG of stormwater on CSO reduction, the MACRO model was used. Considering the functionality of the MACRO model, this technology can be modeled in the same way as near surface storage in the CSSA. The production function for the near surface storage technology was truncated at 30 MG to create the production function for inlet restrictors.

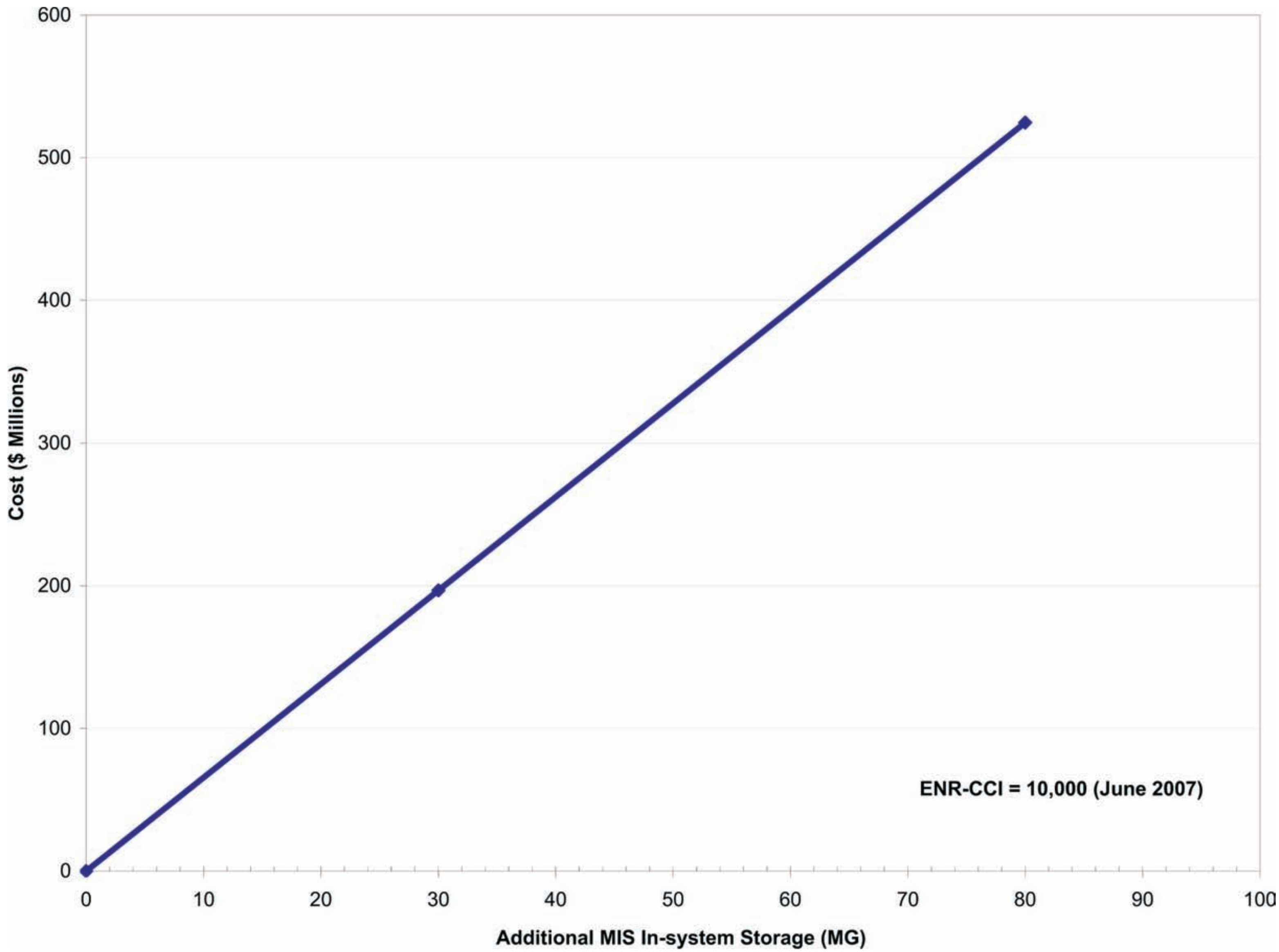

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FIGURE 3A-46

COST FUNCTION: MIS IN-SYSTEM STORAGE FOR 10-FOOT OVERSIZED PIPE IN PLACE OF 4-FOOT PIPE

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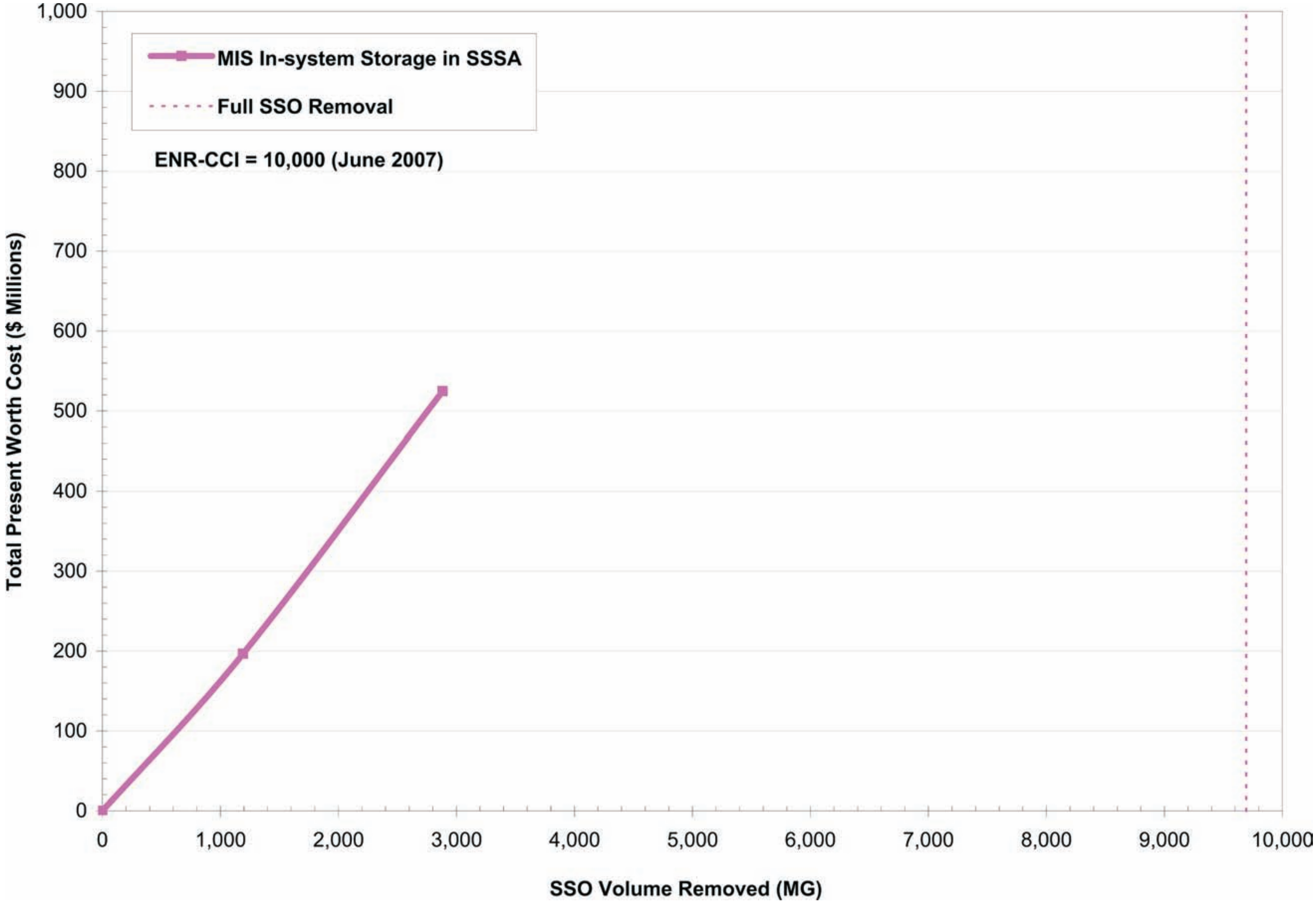

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FIGURE 3A-47
SSO VOLUME COST BENEFIT CURVE: MIS IN-SYSTEM STORAGE
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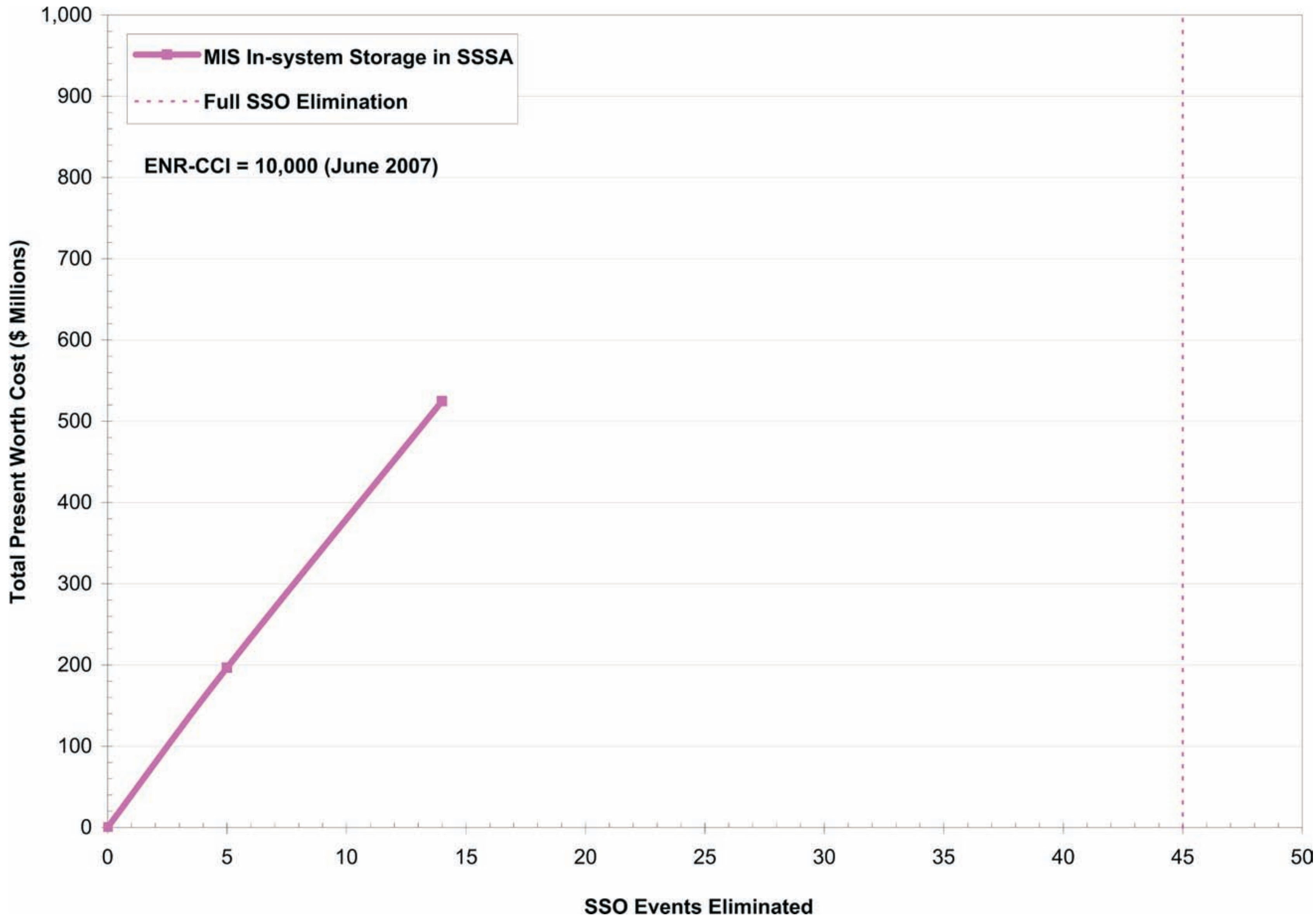

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FIGURE 3A-48
SSO EVENT COST BENEFIT CURVE: MIS IN-SYSTEM STORAGE
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Cost Benefit Curve

Costs for inlet restrictors are highly variable and site specific. This analysis uses the experience from Skokie, IL as an example, but the suitability of these estimates for application in Milwaukee should be carefully studied for the local conditions. A detailed feasibility of applying this technology to Milwaukee will require field investigations and site specific engineering which is beyond the scope of this report.

In Milwaukee, there are 8,600 acres of residential area in the combined sewer service area; it is assumed that all of this area could be served by inlet restrictors. Each block is assumed to use 4 inlet restrictors. If the technology were fully implemented, approximately 6,900 restrictors would be needed to achieve the maximum storage capacity of 30 MG used in the production function above. It is unlikely that the technology could be implemented on each block due to limitations of slope, safety, or residential acceptance.

The total cost is approximately $60 million. This is equivalent to a unit cost of $2.00/gallon. The cost benefit curve for inlet restrictors is shown in Figure 3A-49. The slope of the cost benefit curve is reasonable, but the extent of the curve is limited because the maximum storage volume is 30 MG. It is unlikely that the technology could be implemented fully; consequently, the practical extent of the benefit is limited.

Water stored by inlet restrictors is not prevented from entering the combined sewer system, it is simply delayed. The later timing may coincide with the peak flow from the SSSA and this could add to the SSO problems instead of alleviating them.

3A.2.8 Technology: Rooftop Storage

Introduction

Rooftop storage in the CSSA can temporarily detain stormwater on the roofs of commercial and industrial buildings to reduce the peak flow rate in the combined sewer system. Consequently rooftop storage can reduce CSO volumes. The volume of stormwater that can be stored in this way is limited.

Potential Areas to be Served by Rooftop Storage

Land use information from SEWRPC was used to define the area for potential application of rooftop storage. Land designated as commercial or industrial areas in the CSSA have a total area of 1,555 acres. These areas are designated as non–intensive retail sales and service, manufacturing, and wholesaling and storage. They represent 73% of all the commercial/industrial area in the CSSA.

Rooftop storage was evaluated in the SSSA in an “Ad Hoc” report.(11)

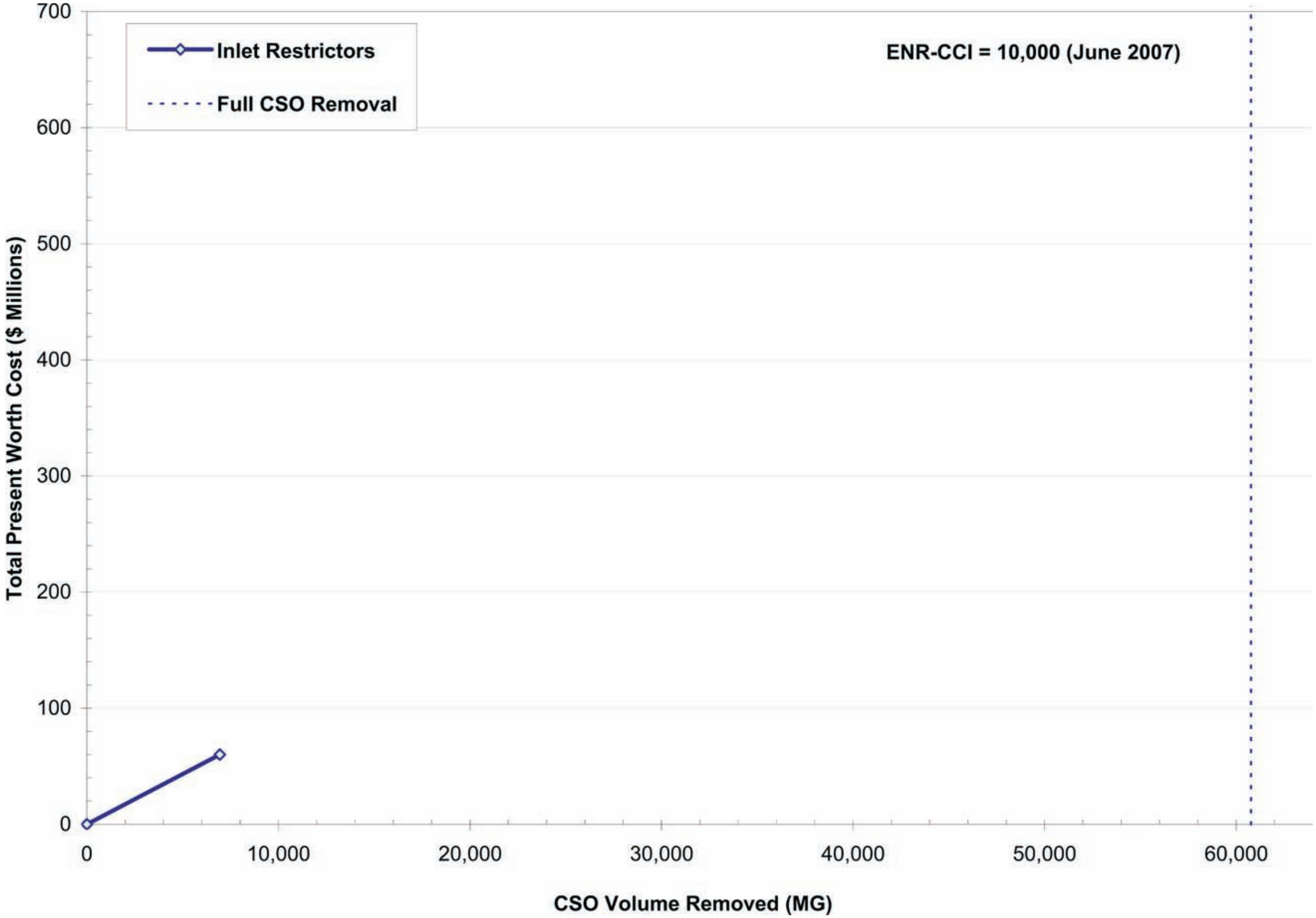

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FIGURE 3A-49
CSO VOLUME COST BENEFIT CURVE: INLET RESTRICTORS
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This study evaluated the benefits of roof storage for the reduction in flow to the sanitary sewer system and the watercourse system. The study focused on roof storage for a "typical" 5.91 acre commercial block. The following assumptions defined the characteristics of roof storage on the buildings of the typical block:

- Buildings occupy 22% of the land area (so the roof area is 57,900 ft^2)
- Only 50% of the roof area is converted to storage
- The design storage depth is 6 inches
- High capacity overflow drains limit the maximum depth to 6 inches

From a structural point of view, it is important to limit the maximum depth to the equivalent water depth of the design snow load. The weight of ponded water should not be greater than the design snow load.

Applying the assumptions of the typical lot to the total area of commercial and industrial lands in the CSSA, the maximum storage volume is estimated to be 28 MG.

Production functions for rooftop storage are the same as the production functions for near surface storage in the CSSA. These production functions were also used for roadway inlet restrictors. The maximum storage volume for rooftop storage (28 MG) is similar to the maximum storage for roadway inlet restrictors (30 MG). Therefore, these two technologies have similar performance.

This technology may be more effective if implemented as a part of a system of stormwater management practices such as downspout disconnection and redirection to a large rain garden or infiltration area. If it is installed as an isolated practice, the detention time of the water on the roof may be too short to be of benefit to the overall system. Using the example in the Ad Hoc report, the outlet orifice diameter is 4 inches. This orifice will drain the roof volume in approximately 10 hours. The water is not prevented from entering the combined sewer system, it is simply delayed. The later timing may coincide with the peak flow from the SSSA and this could add to the SSO problems instead of alleviating them.

The cost of rooftop storage is reasonable if the roof has adequate structural strength. If that is the case the cost is primarily determined by the cost to upgrade the waterproofing of the roof covering. The cost to install flow restrictors on the downspouts is small. Additional cost for maintenance is assumed to be minor. A high capacity alternative outlet will be installed to limit the maximum depth if a restrictor clogs.

It is unlikely that the technology will be implemented if structural modifications are required.

The cost function used information from the Stormtech memorandum.(12)

These cost estimates are approximate; the actual cost may vary considerably depending on the specific conditions of each building. The cost function is based on $5/sq ft for waterproofing costs, $100 per flow restricting device, and one device for each 1,000 sq ft of roof area (based on March 2003 cost information).

The total cost is $49 million (ENR-CCI = 10,000) to store the maximum volume of 28 MG, which is a unit cost of $1.75 per gallon. The cost benefit curve is shown in Figure 3A-50. Like inlet restrictors, the benefits are limited due to the small volume of storage even if implemented to the maximum possible extent.

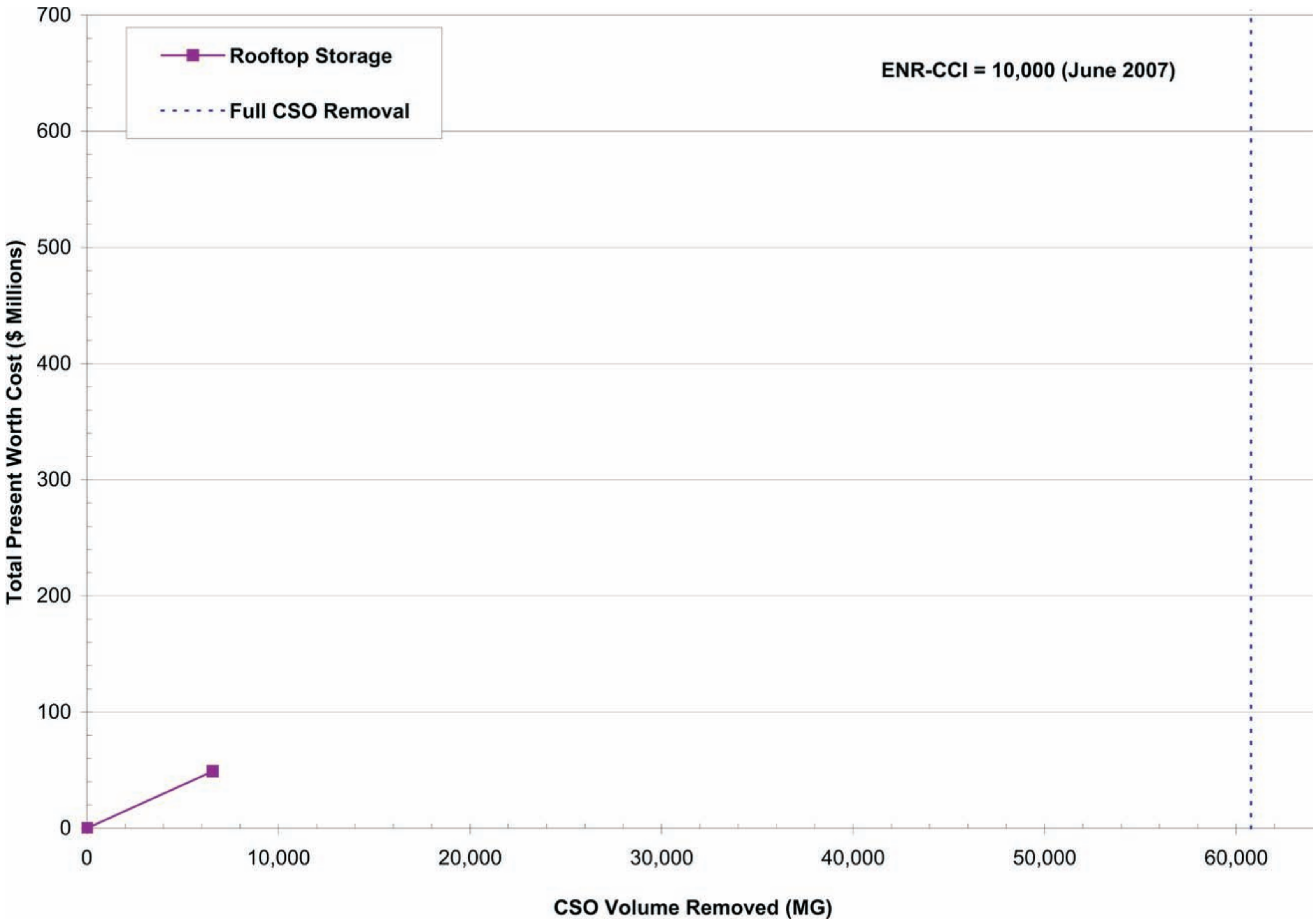

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FIGURE 3A-50
CSO VOLUME COST BENEFIT CURVE: ROOFTOP STORAGE
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The implementation of this technology requires the participation of a very large number of commercial/industrial property owners. The willingness of these owners to participate in this technology could be a significant limitation on implementation of the technology.

3A.3 Wastewater Treatment Plant Technologies

3A.3.1 Technology: South Shore Wastewater Treatment Plant Capacity

Introduction

The SSWWTP plant treats flow from the SSSA and a relatively small amount of flow that is pumped from the ISS. Consequently, additional capacity at SSWWTP benefits SSO reduction directly, but CSO reduction is minor.

The production functions for the CSO and SSO reduction due to increased SSWWTP plant capacity depend on the hydraulic characteristics of the MIS and the ISS pumps.

The cost functions depend on the type of treatment technology and the disinfection method. The wastewater treatment plant technologies identified in Chapter 2, *Technology/Indicator Analysis* were developed into six cost function combinations to be reviewed specifically at SSWWTP and are listed in Table 3A-8. Full secondary treatment can be achieved with either an activated sludge process (AS) or a physical-chemical (ballasted flocculation) process. Blending assumed flow was diverted directly from ISS pumping to disinfection. Disinfection is either by an ultra-violet method (UV) or a chlorine method (Cl). The physical-chemical (ballasted flocculation) process, identified in Chapter 2 as a technology that MMSD evaluated in a separate project from which data were provided, involves the addition of a ballasting agent and chemicals to enhance settling of solids in the wastewater. As presented in this section, the technology is a separate unit process that can be used to treat wastewater during wet weather events.

TABLE 3A-8
COST FUNCTION COMBINATIONS

Cost Function	Treatment Technology	Disinfection Method
1	Full Secondary by Activated Sludge	UV
2	Full Secondary by Activated Sludge	Chlorine
3	Full Secondary by Physical-Chemical	UV
4	Full Secondary by Physical-Chemical	Chlorine
5	Blending	UV
6	Blending	Chlorine

UV =Ultra-violet

Note:
Physical-chemical treatment assumes ballasted flocculation technology

The cost benefit curves are the combination of the four production functions and each of the six cost function curves; a total of 24 cost benefit curves.

Design Factors

In the baseline case, the peak hour treatment capacity at SSWWTP is 300 MGD for full secondary treatment with activated sludge and chlorine disinfection. The baseline case does not include any blending capacity at SSWWTP.

All of the flow to SSWWTP is conveyed by the MIS. It is possible to pump from the ISS to SSWWTP, but pumping is only possible when the ISS gates are closed. The ISS pump discharges into a force main to SSWWTP (the MIS in the vicinity of the KK-1 drop shaft near South 6th Street and Cleveland Avenue); from there it joins the flow in the MIS and is conveyed approximately 12 miles to SSWWTP by gravity.

If the tunnel drop shaft gate at KK-1 is open, the pumped flow would return to the tunnel; consequently, pumping is limited to the times when the drop shaft gate is closed. Pumping is also limited to the time when the bypass gates near DC0103 are also closed otherwise the pumping would increase the SSO volume. Although this pump has a rated capacity of 50 MGD, actual capacity is approximately 40 MGD (see discussion in Section 3A.2.4) and this capacity is only used when there is treatment capacity available at SSWWTP; that is, when the flow in the MIS to SSWWTP is less than the full treatment capacity. Based on MACRO simulations of the baseline condition, the ISS pump to SSWWTP operates an average of 750 hours per year. With an actual 40 MGD pump capacity, the average pumping rate during the periods that the pump is active is 37 MGD.[b]

The maximum possible gravity flow to SSWWTP is approximately 450 to 500 MGD based on the conveyance capacity of the MIS. For the purpose of this analysis, the treatment capacity at SSWWTP is increased from 300 to 480 MGD, which is a range of additional capacity from 0 to 180 MGD. In this range conveyance enhancements are not required, nor is there a need for a dedicated force main to pump flow from the ISS. Therefore, the cost functions were limited to an additional 180 MGD of capacity.

Experience

Wastewater treatment is a proven technology for the reduction of SSOs and CSOs. In combination with tunnel storage, treatment capacity can be used over an extended period of time to reduce both the peak discharge rate and the overall discharge volume of SSOs and CSOs.

Another important benefit of the wastewater treatment plant (WWTP) technology is the number of indicators that are improved by the technology. The WWTP indicators are:

- Volume (CSO and SSO)
- Events (CSO and SSO)
- Total Suspended Solids (TSS)
- Dissolved Oxygen
- E-coli
- Nutrients (nitrogen and phosphorus)

[b] This is described in more detail in Chapter 4, *Watershed Assessment – Existing Conditions* of the *Facilities Plan Report*.

Increasing WWTP capacity provides benefits for multiple indicators; therefore, the benefits of additional treatment are more comprehensive than those of other technologies.

Production Functions

The production functions are based on MACRO simulation results and are useful to define the overall system impact of additional treatment capacity at SSWWTP. MACRO is a simple volumetric model of the MMSD system; it works well when the flow is limited by the treatment plant capacities and the storage volume of the ISS. MACRO is not well suited to cases that are limited by the conveyance capacity of the MIS. MACRO does not have hydraulic routing capabilities. This limitation is not a problem for the baseline case, but as the capacity of SSWWTP increases, the dynamics of conveyance in the MIS become more important. For the case where the treatment capacity of SSWWTP is increased to 480 MGD, the role of conveyance is very important.

The MOUSE model is an alternative to the MACRO model; the MOUSE model is a dynamic hydraulic model of the MIS and the ISS. MOUSE simulation results were used to better define the conveyance dynamics of the MIS when the treatment capacity of SSWWTP was increased to 480 MGD. Using the MOUSE results for guidance, some of the modeling assumptions in MACRO were adjusted. Using these adjusted MACRO assumptions, the production functions for SSWWTP treatment technology were created.

At SSWWTP, the plant capacity is increased from the 300 MGD baseline capacity to 350, 400, and 480 MGD. These represent additional capacities of 50, 100, and 180 MGD greater than the baseline case. The maximum ISS pump out limit to SSWWTP remains at 40 MGD for all the simulations in this case. The increased flow to the plant is conveyed by the MIS and contains wastewater flow from the ISS.

Figure 3A-51 shows the production functions for overflow volume removed by additional treatment at SSWWTP. Figure 3A-52 shows the production functions for overflow events removed by additional treatment at SSWWTP.

Cost Function

The cost function for additional treatment capacity at SSWWTP represents the total costs of applying the technology. It includes the present worth of construction, technical services, O&M, and replacement costs for a 20-year service period. The total costs reflect the present worth as of 2007 (ENR-CCI = 10,000).

The cost function curves, Figure 3A-53, are shown for the three treatment methods, activated sludge, physical-chemical, and blending, and the two disinfection methods, UV and chlorine. The cost functions are plotted against the design peak hourly flow of the additional treatment capacity.

The disinfection method, whether by UV or chlorine, does not impact the cost significantly. The disinfection costs are slightly higher for UV.

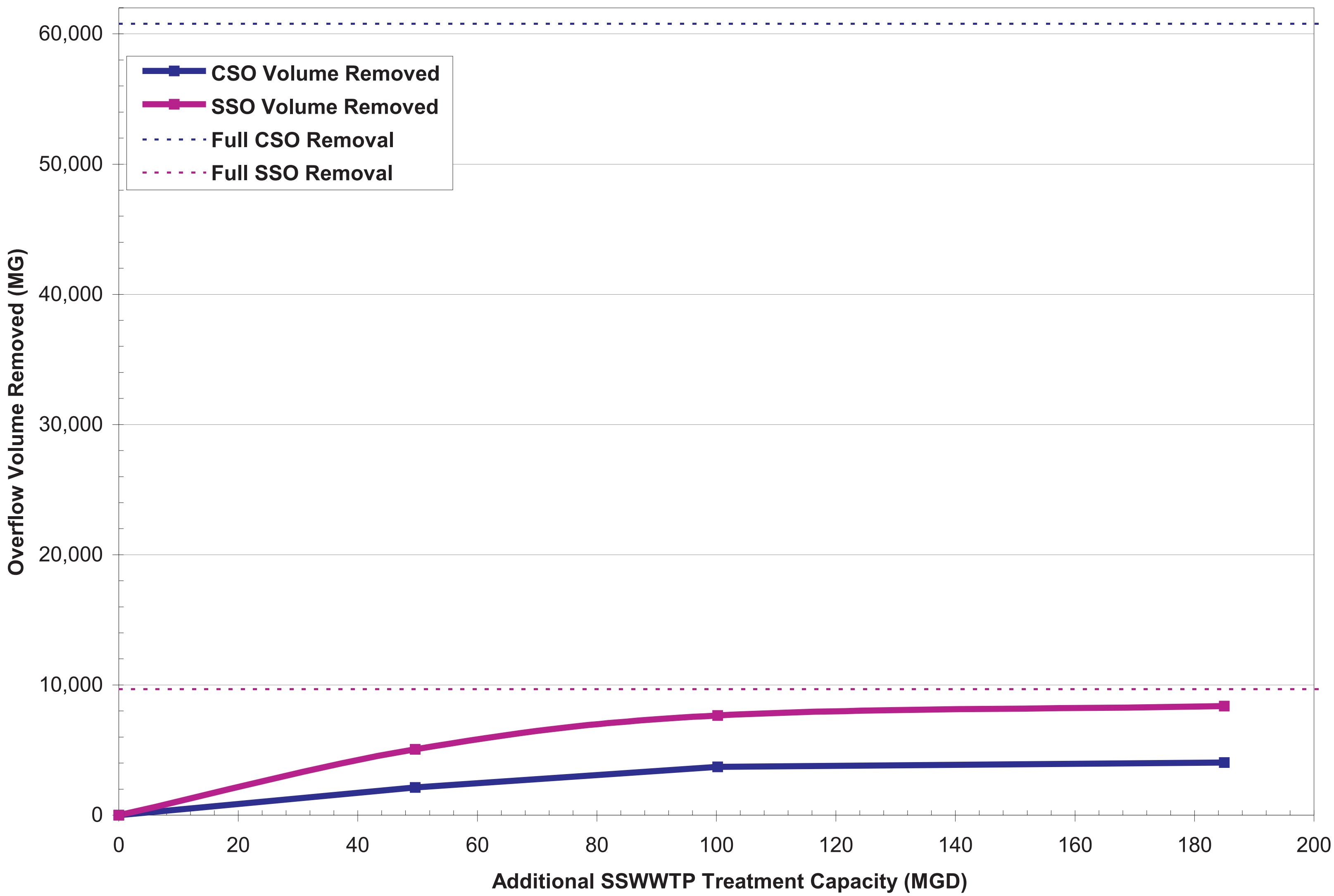

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FIGURE 3A-51
OVERFLOW VOLUME PRODUCTION FUNCTIONS: ADDITIONAL TREATMENT CAPACITY AT SSWWTP
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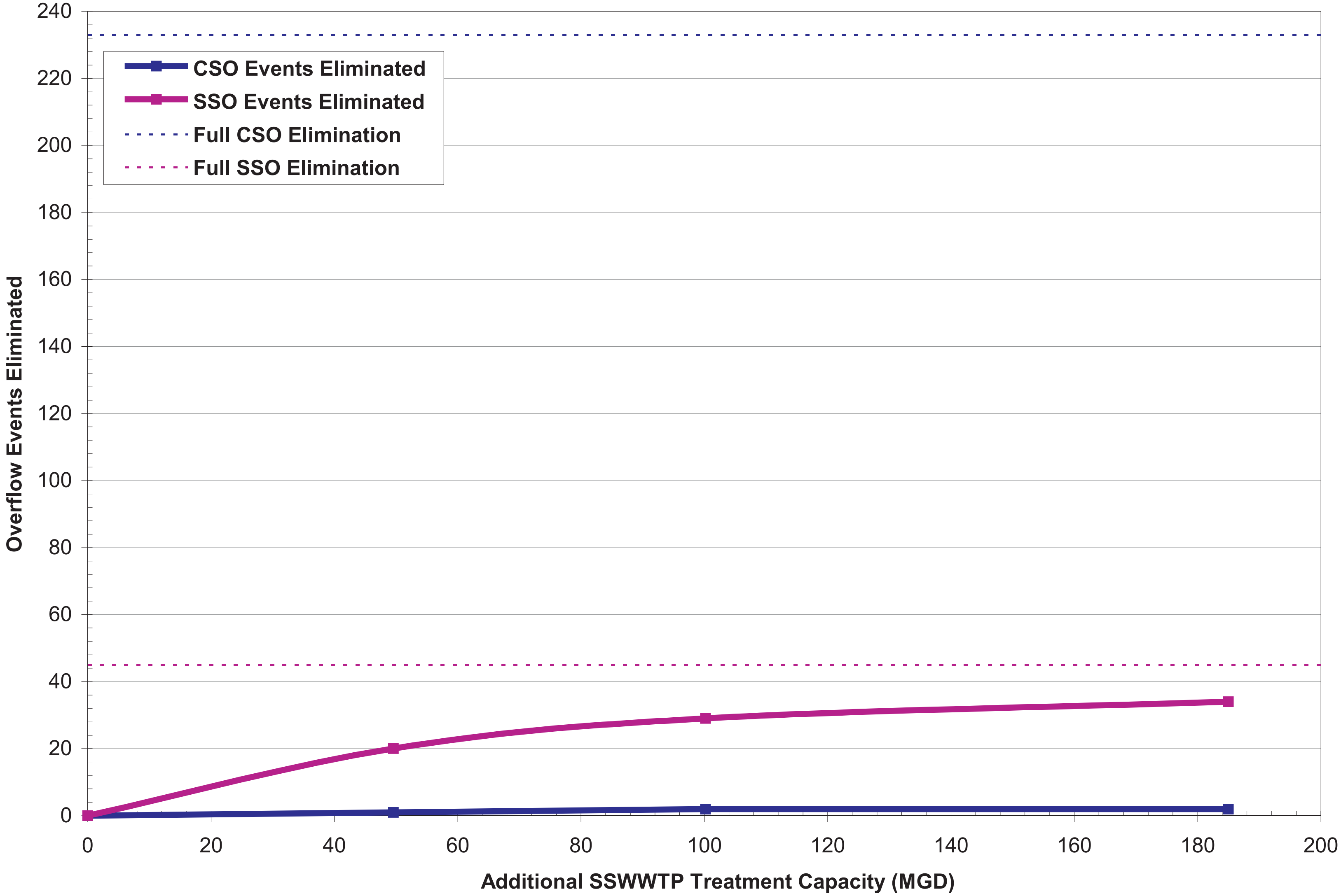

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FIGURE 3A-52
OVERFLOW EVENT PRODUCTION FUNCTIONS: ADDITIONAL TREATMENT CAPACITY AT SSWWTP
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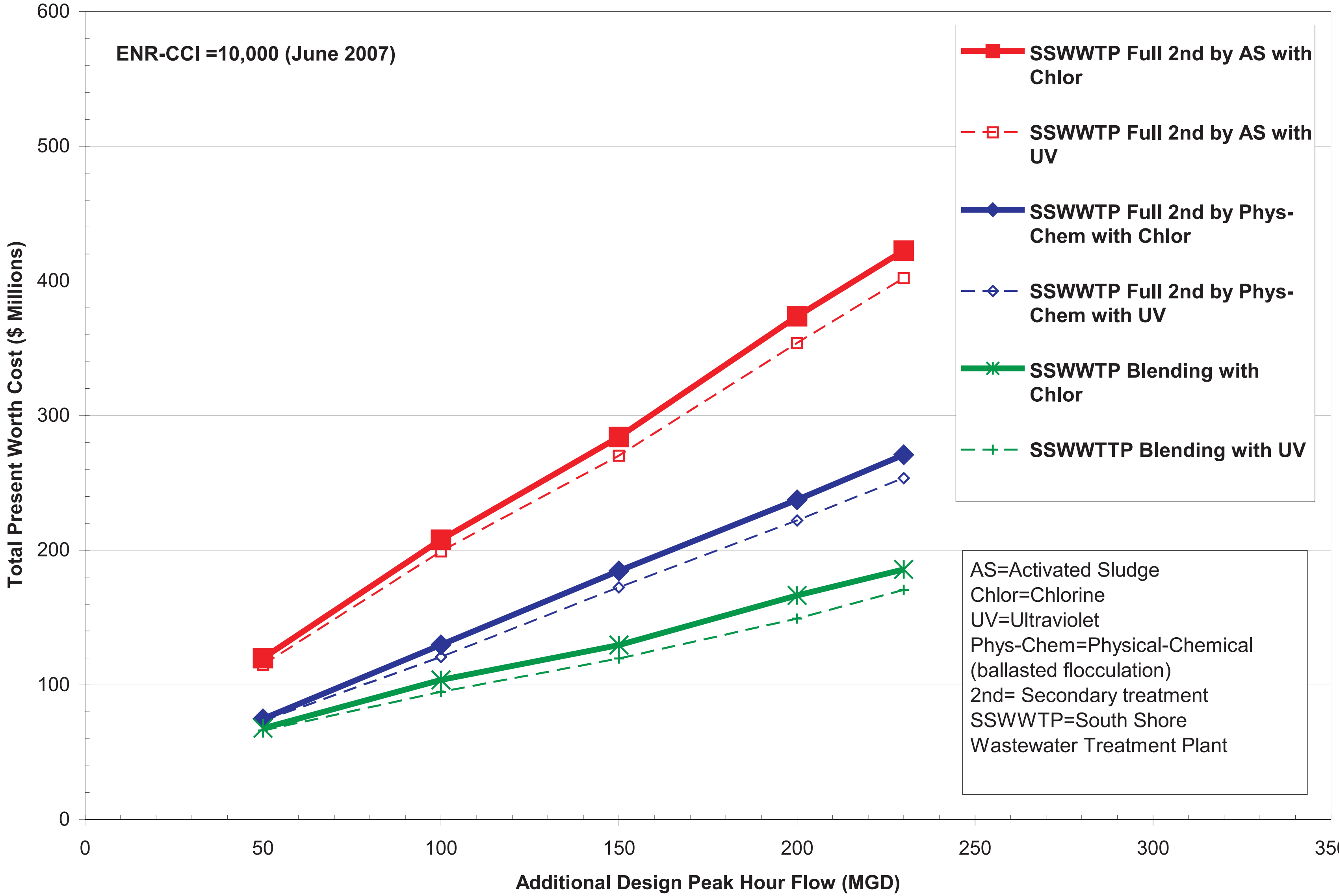

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FIGURE 3A-53

COST FUNCTIONS:
ADDITIONAL SSWWTP CAPACITY

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Cost Benefit Curves: Sanitary Sewer Overflow Indicator

The cost benefit curves were created by integrating the production functions with the cost function. Figure 3A-54 shows the cost benefit curves for SSO volume removed using the three treatment methods and the two disinfection methods. The slope of the cost benefit curves becomes steeper near the end of the curve. Additional treatment capacity up to 150 MGD is cost effective, but further additions to capacity beyond 150 MGD have diminishing benefit.

Figure 3A-55 shows the cost benefit curves for the number of SSO events eliminated.

Cost Benefit Curves: Combined Sewer Overflow Indicator

Additional treatment capacity at SSWWTP achieves SSO reduction more successfully than CSO reduction. The cost benefit curves for CSOs are shown in Figure 3A-56 for CSO volume removed and Figure 3A-57 for CSO events eliminated.

Summary: Additional Capacity at South Shore Wastewater Treatment Plant

Treatment at SSWWTP is a proven and effective technology to reduce SSOs but the benefits for CSO reduction are minor. The existing MIS can convey flow to SSWWTP in the range of 450 to 500 MGD; that is, the range of potential additional treatment capacity is 150 to 200 MGD more (on a peak hour basis) than the baseline capacity.

3A.3.2 Physical-Chemical (chemical flocculation) Secondary Treatment Process

The innovative physical-chemical secondary treatment option developed for treatment of wet weather flows that is described in this section is chemical flocculation, which is a process that uses existing primary clarifiers and newly developed chemical feed schemes and process modifications. It is a different option than the physical-chemical (ballasted flocculation) process described in Section 3A.3.1 and is a variation of the chemical enhanced primary technology originally identified in Chapter 2, *Technology/Indicator Analysis*. This section describes the technology, basis for design, process, sequence of operations during wet weather, major equipment, anticipated system performance, conceptual cost estimates, and production theory cost benefit analysis. This process was only considered at SSWWTP because it would be more cost effective to implement there than at JIWWTP.

The MMSD is in the process of completing a preliminary engineering study for the implementation of physical-chemical (ballasted flocculation) secondary treatment at SSWWTP. When this study is complete, the analysis will supplement the evaluation of physical-chemical treatment in the 2020 FP.

Background

Physical-chemical (chemical flocculation) secondary treatment is the process by which raw sewage is conditioned with chemicals, typically metal salts and/or polymers, to improve liquid/solid separation. This process has been applied in Norway since the early 1970s with reported biochemical oxygen demand (BOD) removals of 70% without biological treatment.

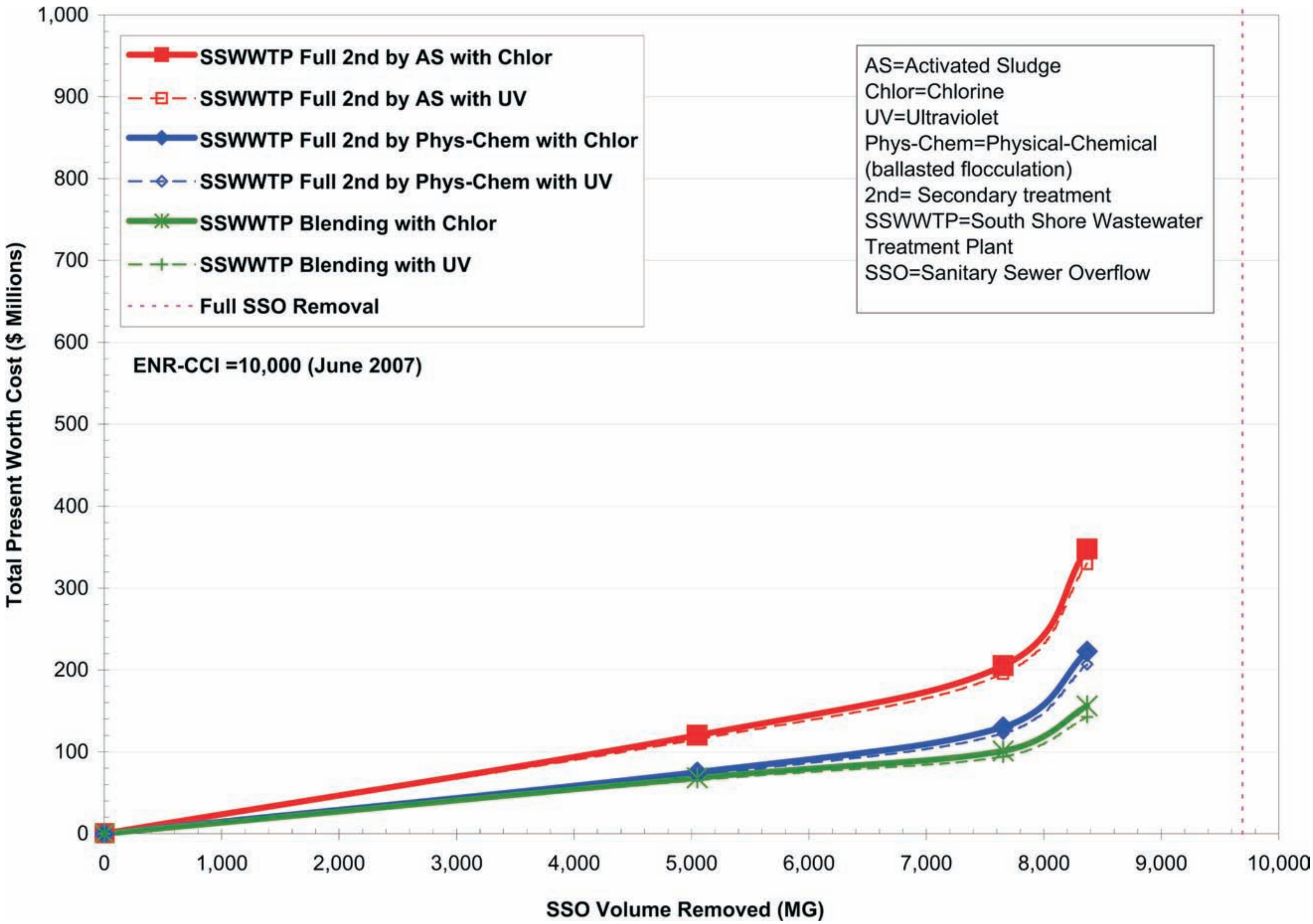

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FIGURE 3A-54

SSO VOLUME COST BENEFIT CURVES: ADDITIONAL TREATMENT AT SSWWTP

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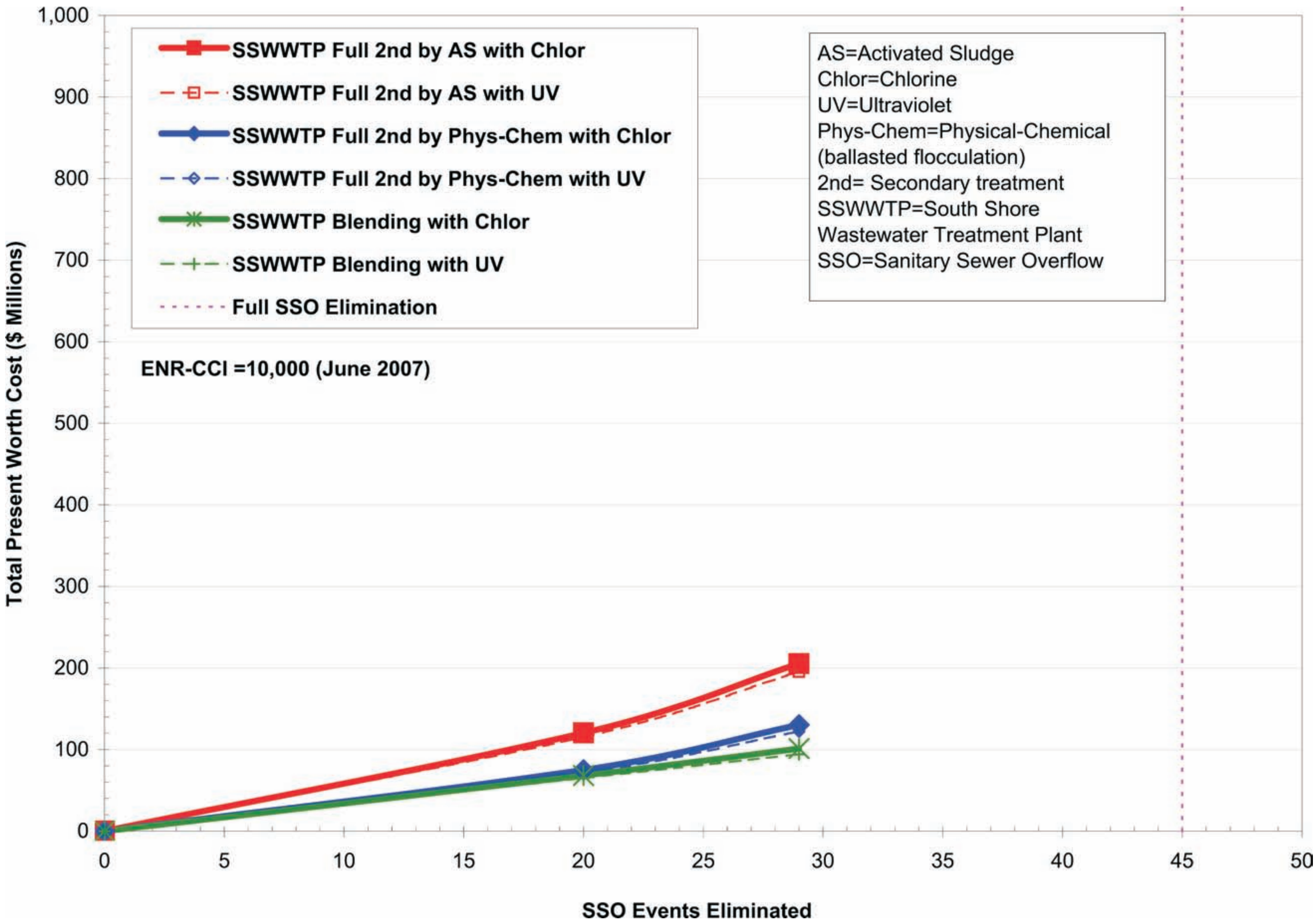

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FIGURE 3A-55
SSO EVENT COST BENEFIT CURVES: ADDITIONAL TREATMENT AT SSWWTP

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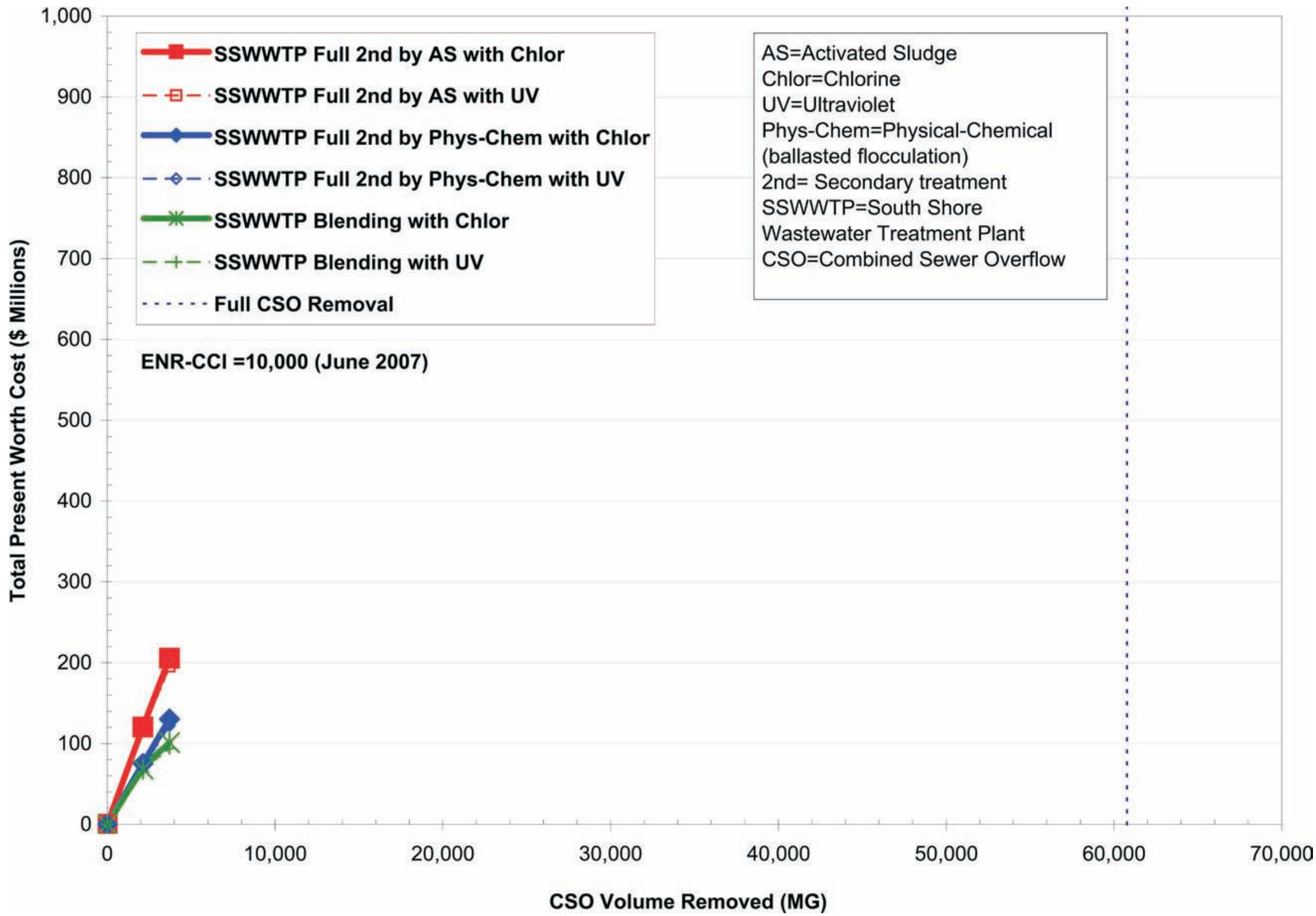

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FIGURE 3A-56

CSO VOLUME COST BENEFIT CURVES: ADDITIONAL TREATMENT AT SSWWTP

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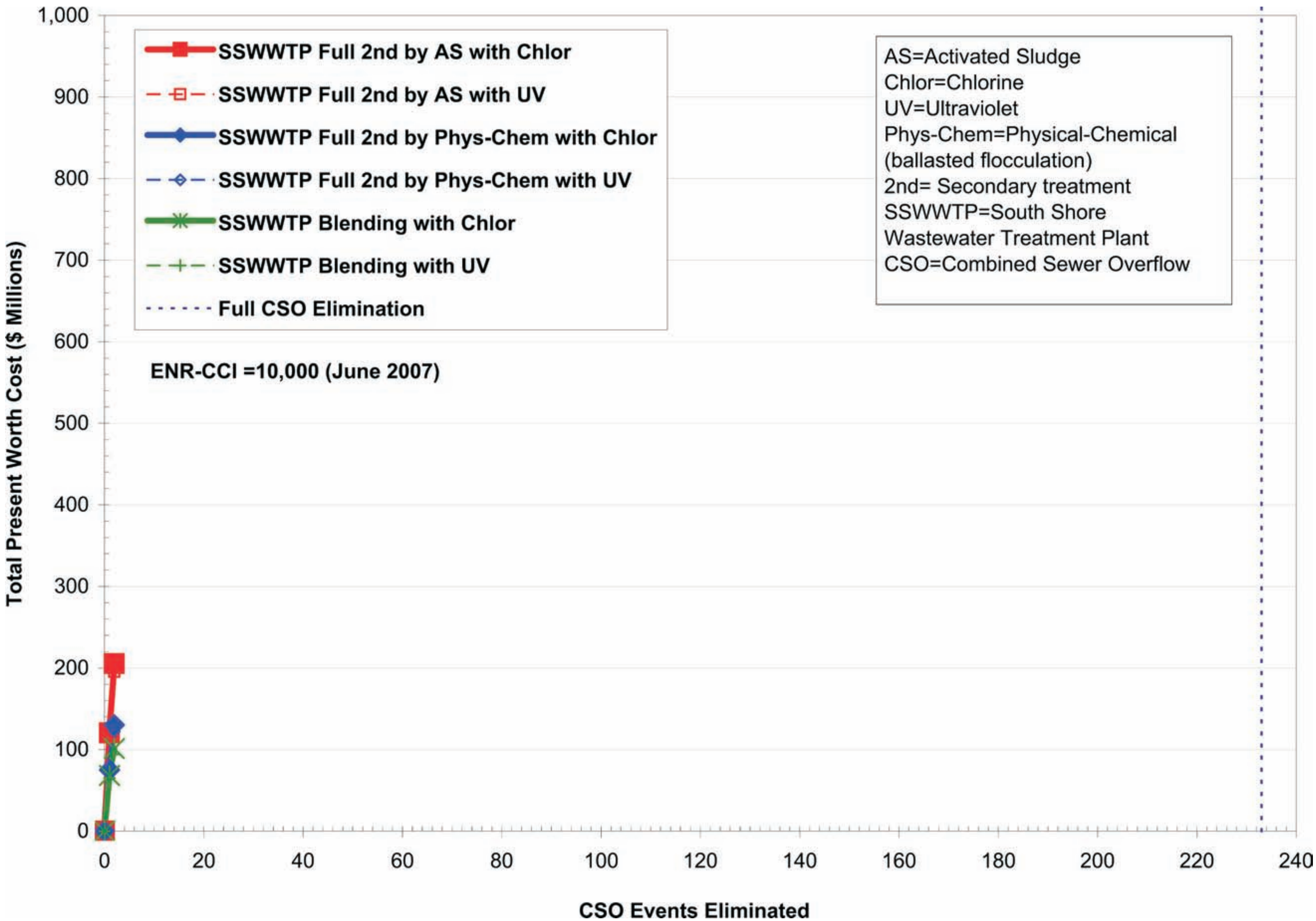

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FIGURE 3A-57

CSO EVENT COST BENEFIT CURVES: ADDITIONAL TREATMENT AT SSWWTP

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To begin the screening process for application in the 2020 FP, jar testing[c] was conducted between May 9 and May 13, 2006 on wastewaters received at SSWWTP to screen likely coagulants (ferric chloride ($FeCl_3$) and polyaluminum chloride (PACl) and flocculants (Ciba anionic polymers) and to observe treatment performance. The control for comparison was wastewater that did not receive chemical conditioning. During the testing period a total of 1.5-inches of rain fell in the collection area – so the wastewater tested reflected wet weather conditions. Testing results are summarized in Tables 3A-9 and 3A-10.

The use of flocculant-assisted single or dual coagulant chemistries increased percent TSS removals from the upper 60s (control) to the mid-90s and reduced effluent TSS concentrations from 31 to 4 mg/L. See Table 3A-9.

There were similar dramatic improvements in BOD removal with overall percent BOD removal increasing from 65% to more than 85%. See Table 3A-10. Although soluble BOD (sBOD) removal was observed, this is merely a function of colloidal BOD removal rather the true soluble BOD removal. Chemical coagulation processes will primarily remove the particulate (pBOD) fraction as shown in Table 3A-10. The chemically conditioned effluent BOD concentration was significantly lower (12 mg/L) compared to the control at 29 mg/L.

Bench scale and full scale physical-chemical chemical flocculation testing was conducted in Seattle, WA at King County's South Plant in 2005 to support the design for the process at the new King County (WA) Brightwater Treatment Plant. In the 2005 testing, favorable bench-scale results were a positive indicator for the full-scale performance. Extrapolation of bench-scale test data to full-scale plant performance at King County's South Plant indicates that wet weather flows directed to the South Shore facility are amenable to chemical treatment and that gains in full-scale capacity are likely achievable.(13)

TABLE 3A-9
SUMMARY OF SUSPENDED SOLIDS REMOVAL

	Control	$FeCl_3$ w/ Polymer	$FeCl_3$ & PACI w/ Polymer
Influent TSS (mg/L)	99	99	99
TSS after 15 min Settling (mg/L)	31	4	3
TSS Removal Efficiency (%)	69	96	97
Influent VSS (mg/L)	85	85	85
VSS after 15 min Settling (mg/L)	28	4	2
VSS Removal Efficiency (%)	67	95	98

$FeCl_3$ = Ferric Chloride
PACI = Polyaluminum Chloride
TSS = Total Suspended Solids
VSS = Volatile Suspended Solids

[c] Jar testing is a pilot-scale test of treatment chemicals used in a water treatment process. It helps to determine the correct amount of treatment chemicals to use in full-scale treatment plant operations.

TABLE 3A-10
SUMMARY OF BIOCHEMICAL OXYGEN DEMAND REMOVALS

	Control	$FeCl_3$ w/ Polymer	$FeCl_3$ & PACl w/ Polymer
Influent BOD (mg/L)	83	83	83
BOD after 15 min Settling (mg/L)	29	12	8.8
BOD Removal Efficiency (%)	65	86	89
Influent sBOD (mg/L)	13	13	13
sBOD after 15 min Settling (mg/L)	13	9.6	8.3
sBOD Removal Efficiency (%)	0	26	36
Influent pBOD (mg/L)	70	70	70
pBOD after 15 min Settling (mg/L)	16	2.4	0.5
pBOD Removal Efficiency (%)	77	97	99

$FeCl_3$ = Ferric Chloride
PACl = Polyaluminum Chloride
sBOD = Soluble Biochemical Oxygen Demand
pBOD = Particulate Biochemical Oxygen Demand

Basis for Evaluation

The evaluation is based upon additional preliminary (screening and grit removal) and primary (sedimentation) treatment at SSWWTP to meet the peak flow projections through the year 2020. The SSWWTP will require the capacity to process a peak flow of 450 million gallons per day (MGD) during wet-weather events when infiltration and inflow significantly increase the volume of wastewater requiring treatment. Current process limitations give SSWWTP a capacity of 300 MGD. Therefore, the proposed modifications will accommodate 450 MGD through the preliminary and primary facilities, but no modifications of the secondary treatment facility (activated sludge process) are considered in this plan. The secondary facility has successfully processed flows of 300 MGD, so the excess flow from the preliminary and the revised physical-chemical systems in the primary clarifiers (any flow above 300 MGD) will be routed directly to a dedicated UV disinfection system, monitored, and then recombined with the secondary activated sludge process effluent for discharge into Lake Michigan.

The basis of evaluation is predicated on the following conditions:

- Maximum $FeCl_3$ dosage is 50 mg/L
- Maximum PACl dosage is 20 mg/L
- Maximum polymer dosage is 0.5 mg/L
- Design flow is 450 MGD
- Peak flow duration is 7 days
- Maximum primary sludge withdrawal rate is 2,000 gpm

A generalized block flow diagram for the physical-chemical (chemical flocculation) modifications to SSWWTP is presented in Figure 3A-58.

An alternative approach considered was to take advantage of the low-strength nature of SSWWTP wastewater. During rain events, based upon available data, the primary influent characteristics are 100 mg/L BOD and 102 mg/L TSS. If 300 MGD of this wastewater were to be routed through preliminary treatment and then directly to the activated sludge process, the overall loadings would not exceed the activated sludge process capacity. This secondary treatment unit process arrangement is permissible under NR 110.21(3)(c), and is essentially the same unit process system that existed at JIWWTP until the 1980s. This would free the chemical flocculation system to process the remaining 150 MGD under much lower loading conditions. This combined secondary treatment with activated sludge and physical-chemical treatment would produce a high quality effluent. This approach is presented in Figure 3A-59.

Both alternatives offer MMSD significant gains in capacity with minimal new construction. Another benefit is that these alternatives would use equipment commonly operated by the plant staff rather than install new unit operations that would be rarely used and therefore less familiar to the operators.

The first alternative was selected for financial analysis because it will have the highest capital and operating costs. The final determination of the process to be constructed would be made after the completion of a preliminary engineering analysis. Full-scale testing (demonstration plant) is recommended. It will be beneficial and worth the cost involved for optimizing the design and determining the overall performance.

Process

The block flow diagram presented in Figure 3A-59 and the aerial photo with superimposed graphic in Figure 3A-60, show the process as follows:

1) Wastewater will enter the facility through the existing flow control structure
2) A new channel and associated gates will be installed to allow 150 MGD to be directed to a new preliminary treatment area. The initial channel will divide the flow equally into three trains, each containing a screen and a grit tank of like design to the existing seven trains. The residuals (screenings and grit) will be managed with identical systems as are currently in place at SSWWTP.
3) Between the screens and the grit tanks in both the new and existing process trains, provisions will be made to add $FeCl_3$ in the lead position and PACl immediately downstream. Levels in the grit tanks will be maintained by a single downstream flow control valve as is the current practice.
4) Effluent from the new preliminary facility will be routed to the southern end of the primary clarifier feed channel to a flow control box with overflow weirs. At this point the flow will be split evenly to the four banks of primary clarifiers.
5) Polymer will be added through an injection quill to the pipe feeding the flow control box.
6) From the existing grit effluent channel, the flow will be evenly split through the four existing flow meters and directed to the 16 existing primary clarifiers.

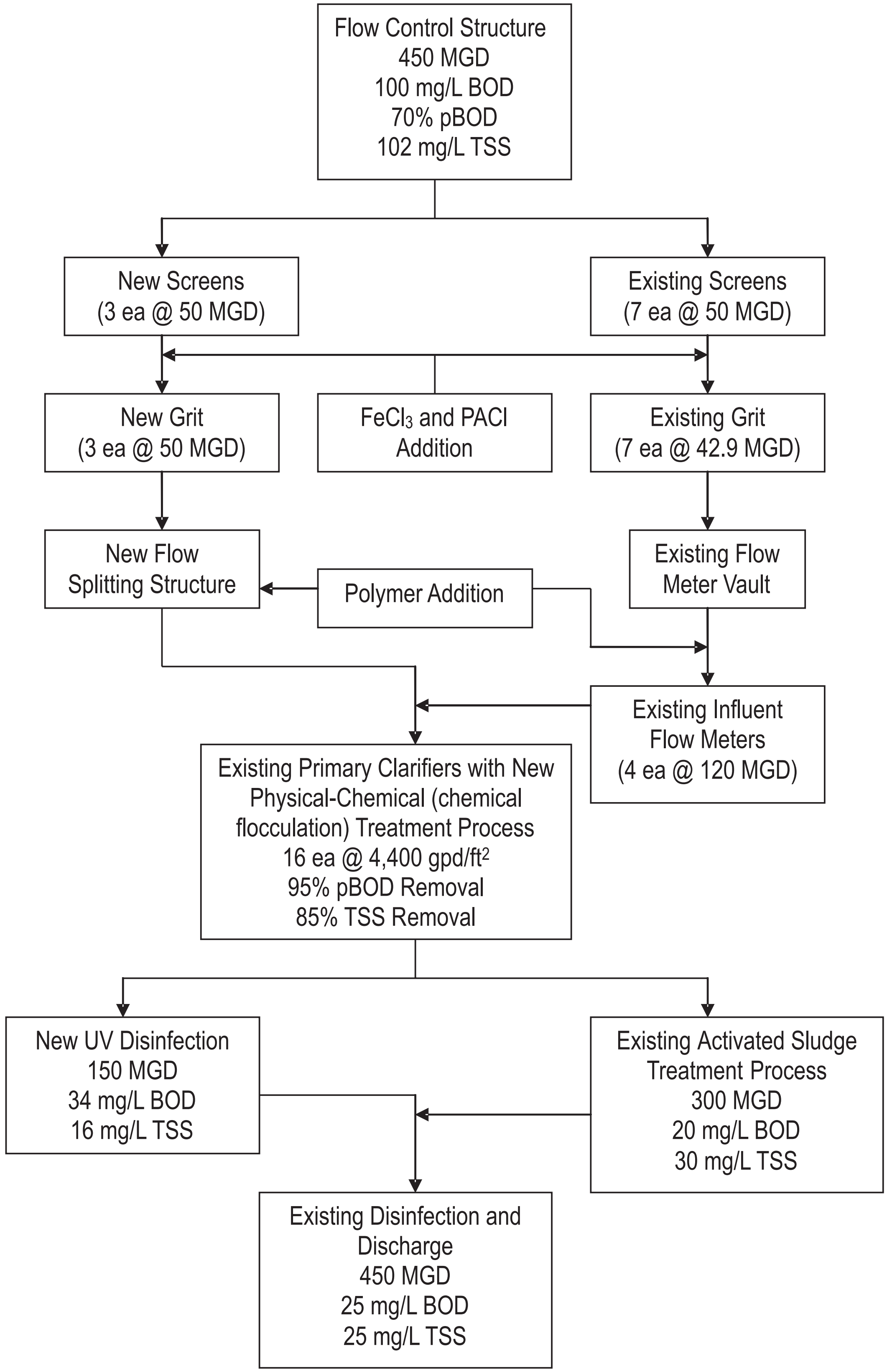

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FIGURE 3A-58

MODIFIED BLOCK FLOW DIAGRAM FOR THE NEW PHYSICAL-CHEMICAL (CHEMICAL FLOCCULATION) TREATMENT PROCESS MODIFICATION TO SSWWTP

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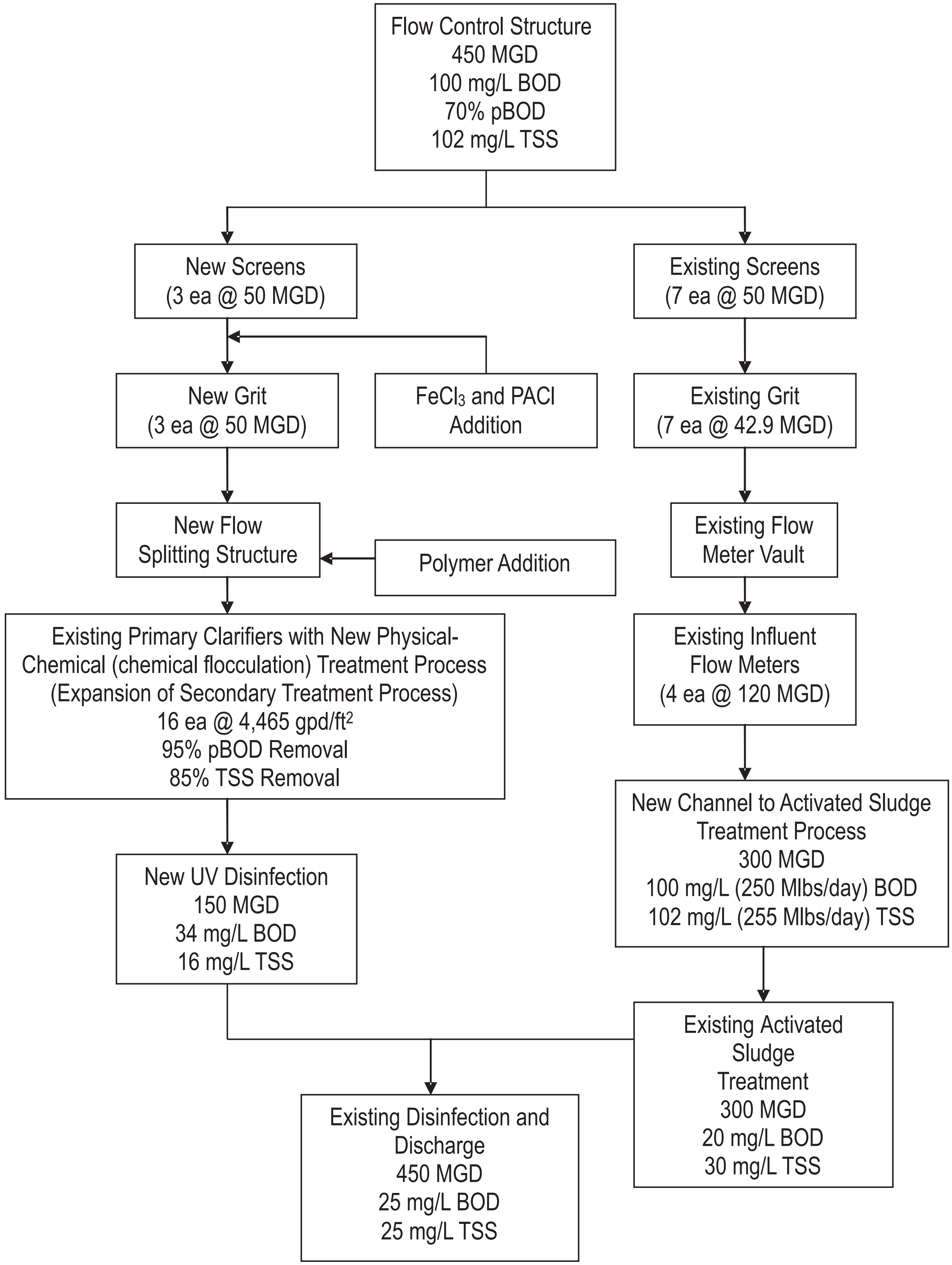

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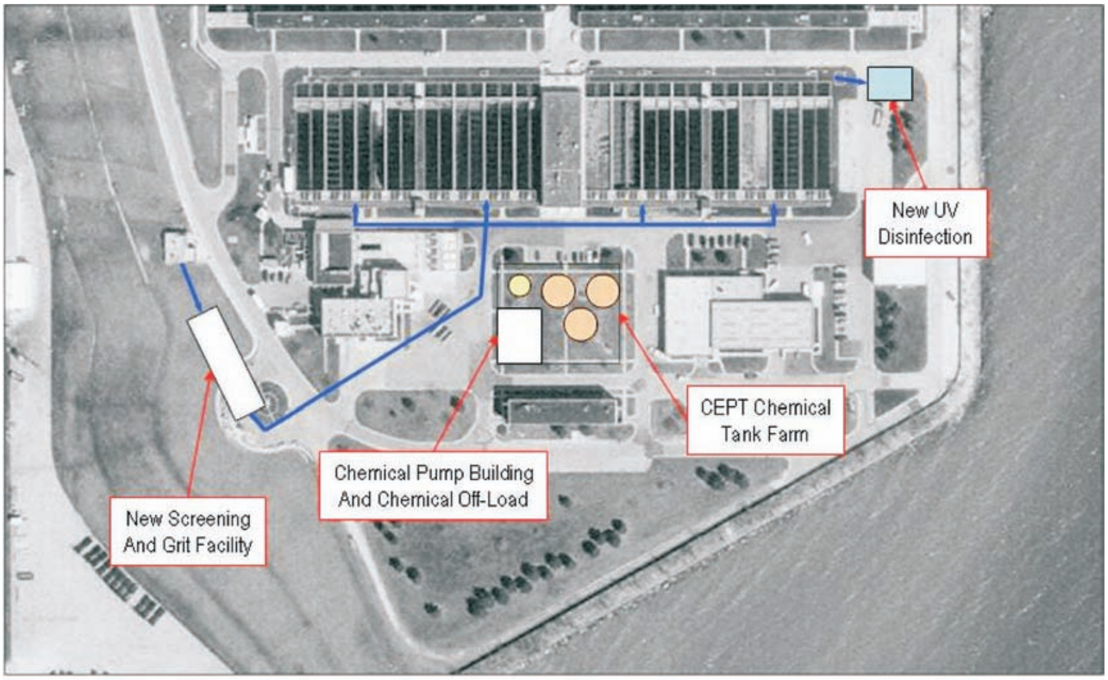
FIGURE 3A-59

ALTERNATIVE MODIFIED BLOCK FLOW DIAGRAM FOR THE NEW PHYSICAL-CHEMICAL (CHEMICAL FLOCCULATION) TREATMENT PROCESS MODIFICATION TO SSWWTP



UV=Ultraviolet
CEPT=Physical-chemical (chemical flocculation)

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FIGURE 3A-60
AERIAL VIEW OF SSWWTP WITH PROPOSED PHYSICAL-CHEMICAL (CHEMICAL FLOCCULATION) TREATMENT MODIFICATIONS

7) At the discharge of the flow meter vault, polymer will be injected through a quill into each pipe (a total of four pipes) leading to a bank of four primary clarifiers (a total of 16 clarifiers).
8) Primary clarifiers will operate without physical modifications, but the use of chemical coagulants and flocculants will be incorporated, which will increase the allowable surface overflow rate (SOR) of the primary clarifiers, expanding the installed capacity from 300 MGD to 450 MGD.

   A new flow control structure will be added to the east side of the existing primary effluent channel to allow all physical-chemical secondary effluent flow in excess of 300 MGD to be routed directly to the new UV disinfection system.
9) Primary sludge will be removed using the existing sludge pumps and routed to the anaerobic digester as is the current practice. The new UV disinfection system will be sized for 150 MGD peak flow and be located immediately to the east of the primary clarifiers.
10) The disinfected effluent will be discharged to the existing secondary treatment bypass channel, where it will be sampled and routed to the effluent diffuser.

Sequence of Operations during Wet Weather

Physical-chemical (chemical flocculation) secondary treatment is intended to be used during high-flow wet-weather events. As projected flows increase above 250 MGD, the operators would bring the new physical-chemical treatment system online at a target flow rate (initially set to 150 MGD). As influent wastewater flows exceed 300 MGD, the operators would begin flow-paced addition of coagulants and flocculant. Dosage adjustments are not automated. An operator-monitored process performance can adjust dosages using a manual control. Online turbidity meters will report primary effluent quality allowing the operator to adjust chemical dosages accordingly.

When flows exceed 300 MGD, the operators will activate the UV disinfection system and begin routing physical-chemical (chemical flocculation) secondary treatment effluent to the UV system. The intention is for the activated sludge process to operate at a constant 300 MGD with all additional flow being directed to the UV disinfection system.

As flows drop below 300 MGD, the operators will deactivate the UV disinfection system. Primary effluent contained in the system will drain back to the activated sludge process. When out of service, the UV system will be drained and cleaned.

As flows drop below 250 MGD, the operators will deactivate the physical-chemical (chemical flocculation) treatment system and take the new preliminary system off-line. Chemical inventories will be recorded and adjusted as necessary in preparation for the next event.

Physical-chemical (chemical flocculation) Equipment

The required equipment for physical-chemical (chemical flocculation) secondary treatment and preliminary sizing are presented in Table 3A-11.

System: Screening	(Duplicate existing)
Total Capacity (MGD)	150
Type	Parkson self-cleaning
Number	3
Size (ft)	5 wide by 15 high
Capacity (MGD each)	50

System: Grit Removal	(Duplicate existing)
Total Capacity (MGD)	150
Number of Channels	3
Depth (ft)	15.25
Width (ft)	12 at top, 6 at bottom
Length (ft)	120
Type of Collectors	Chain and flight
Grit Pump Capacity	3 @ 600 gpm each
Grit Classification Type	SlurryCup
Capacity	3 @ 590 gpm each

System: $FeCl_3$ Addition	(New facility)
Dosage (mg/L)	30 to 50
Total Storage Tank Volume (gal)	300,000
Number of Storage Tanks	3
Tank Diameter (ft)	26
Number of Feed Pumps	10 Operating, 2 Standby
Feed Pump Capacity (gpm)	1 to 5 each
Feed Pump Type	Chemical Metering Pump
Chemical Diffusers	10

System: PACl Addition	(New facility)
Dosage (mg/L)	10 to 20
Total Storage Tank Volume (gal)	60,000
Number of Storage Tanks	3
Tank Diameter (ft)	18
Number of Feed Pumps	10 Operating, 2 Standby
Feed Pump Capacity (gph)	12 to 30 each
Feed Pump Type	Chemical Metering Pump
Chemical Diffusers	10

System: Polymer Addition	(New facility)
Dosage (mg/L)	0.3 to 0.5
Number of Totes	5
Volume Neat for 7 Days (gal)	1,500
PolyBlend Units	10 Operating, 2 Standby

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TABLE 3A-11
PHYSICAL-CHEMICAL (CHEMICAL FLOCCULATION) EQUIPMENT

In addition to the equipment identified above, a stand-alone UV disinfection system will be provided for the treatment of up to 150 MGD of effluent from the physical-chemical treatment process. As with the new screening and grit processes, sizing and costs developed previously are used in this evaluation.

Anticipated System Performance

The dry weather wastewater currently processed at SSWWTP is considered dilute by industry standards in terms of average BOD and TSS concentrations (see comparison of JIWWTP and SSWWTP in *Treatment Report,* Chapter 4). During wet weather events this wastewater is further diluted with infiltration and inflow from the collection system.

At a design flow of 450 MGD with all 16 primary clarifiers in service, the surface overflow rate (SOR) will be 4,400 gpd/ft^2 (4,700 gpd/ft^2 with one clarifier out of service). These rates are consistent with those at other physical-chemical secondary treatment facilities. Full-scale physical-chemical treatment trial data from King County's South Treatment Plant indicated TSS removal efficiencies of 65-85% at an SOR of 5,000 gpd/ft^2. The BOD removal is directly correlated to the particulate BOD (pBOD) in the wastewater. During the jar testing at SSWWTP, virtually all the pBOD was removed, which was expected given the high TSS removals observed.

Without full scale equipment specific testing it is difficult to predict full-scale performance based on jar testing alone. In order to better estimate anticipated system performance, full-scale demonstration testing at SSWWTP is recommended. This testing could be conducted by modifying a single battery of primary clarifiers for physical-chemical (chemical flocculation) treatment. The test should be of sufficient length to verify performance at extended peak hydraulic loadings. This would allow the system to be stressed and resulting performance evaluated. A temporary system would be installed and staffed to evaluate physical-chemical (chemical flocculation) treatment under a variety of storm flow events. Data collected during this test would allow a more rigorous evaluation of physical-chemical (chemical flocculation) treatment specific to SSWWTP, including but not limited to the chemical dosages, application points, clarifier performance, residuals generation, and specific water quality for UV disinfection performance. The cost for a thorough study of physical-chemical (chemical flocculation) treatment would range between $0.75 and $1.5 million depending upon the study duration and staffing.

Conceptual Cost Estimates

Cost estimates were developed using specific equipment requirements for physical-chemical (chemical flocculation) secondary treatment and common costs previously developed for the modifications to the screening, grit, and UV disinfection systems. Specific assumptions used for the cost estimate of the physical-chemical (chemical flocculation) treatment modifications included:

- No demolition required for chemical storage facilities
- Chemical storage tanks can be located as indicated in Figure 3A-60
- Chemical storage volume of seven days is required
- Chemical storage facilities can be located as indicated in Figure 3A-60

- Sufficient head is available to direct flow from flow control structure, through new preliminary treatment, and to the new flow control box
- Chemical feed piping will be heat traced and insulated

Table 3A-12 presents a summary of capital costs associated with the implementation of physical-chemical (chemical flocculation) secondary treatment at SSWWTP.

TABLE 3A-12
PHYSICAL-CHEMICAL (CHEMICAL FLOCCULATION) TREATMENT (ONLY)
FACILITY COST ESTIMATE SUMMARY

Item	Expense ($ M)	Basis
Screening/Grit Facilities	$ 13.31	Scaled from TM-5, Preliminary Engineering for HRT[1]
Conveyance and Controls	3.82	Tie ins to existing MIS, flow meters and valves for new preliminary system, piping and flow control structure at primary clarifiers
Ferric Chloride Addition System	0.41	Storage tanks, chemical metering pumps, air induction diffusers
Polyaluminum Chloride Addition System	0.20	Storage tanks, chemical metering pumps, air induction diffusers
Polymer Addition System	0.09	Dry chemical make-up system, storage tanks, chemical metering pumps, air induction diffusers
Chemical Storage Building and Truck Off-Load	0.36	Temperature controlled building with chemical truck unloading system
UV Disinfection System	12.80	Scaled from TM-5, Preliminary Engineering for HRT[1]
Total Facility Cost	**$ 30.99**	

HRT = High Rate Treatment (definition used in the report is physical-chemical treatment)
MIS = Metropolitan Interceptor Sewer

Source: 1) Evaluation and Preliminary Engineering for Chemically Enhanced Clarification of Wet Weather Flows, Conceptual Design, Draft Report August 2006

The annual expected O&M costs were developed in accordance with the following conditions:

- Labor cost of $36.06/hour
- Three operators per event
- Four events per year
- Each event averages three days in duration
- Average annual volume requiring chemical conditioning is 847 MG
- Chemical dosages default to peak expected concentrations

- Ferric chloride quoted at \$0.22/dry lb
- PACl quoted at \$0.30/dry lb
- Polymer quoted at \$2.00/dry lb
- Power based on 1,000 hp installed at \$0.0431/kW·hr
- Maintenance at 3% of total equipment cost
- 25% contingency to address uncertainties

The resulting anticipated annual O&M costs are \$1,350,000 (ENR-CCI = 10,000).

Using the costs for physical-chemical (chemical flocculation) treatment and the annual O&M developed above, a total present worth cost was developed for this alternative. Included in this cost are facilities for effluent pumping and a second outfall diffuser pipe into Lake Michigan because the existing outfall system may not have enough capacity if this system would be implemented. All markups and adjustments made for this analysis are consistent with those used previously, allowing a direct comparison between the physical-chemical (chemical flocculation) treatment analysis and the other high rate treatment options and are presented in Table 3A-13 below. The lifecycle cost analysis is summarized in Table 3A-14.

While physical-chemical (chemical flocculation) treatment may prove an economically attractive alternative for MMSD, the performance cannot be confirmed without full-scale (demonstration plant) physical-chemical (chemical flocculation) treatment testing, and therefore should only be advanced with a commitment to conduct the testing.

Physical-Chemical Treatment (chemical flocculation)			$14,369,000
Disinfection - **UV**			$12,800,000
Yard Piping			$3,821,000
Effluent Pumping			$5,447,000
Outfall			$9,905,000
		Subtotal	**$46,342,000**
Electrical and I & C (15%)	15%		$6,951,000
Piping (1%)	1%		$463,000
Yardwork (0.1%)	0.10%		$50,000
		Subtotal	**$53,800,000**
Mobilization/Demobilization (7%)	7%		$3,800,000
Land			$0
		Subtotal	**$57,600,000**
Contingencies (25%)	25%		$14,400,000
		TOTAL ESTIMATED CONSTRUCTION COST	**$72,000,000**
Non-Construction Cost (35%)	35%		$25,000,000
TOTAL ESTIMATED PROJECT COST, 6/07 Present Worth			**$97,000,000**
		PROJECT COST / GALLON	**$0.65**

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TABLE 3A-13

CAPITAL COST DEVELOPMENT FOR PHYSICAL-CHEMICAL (CHEMICAL FLOCCULATION) TREATMENT AT SOUTH SHORE WASTEWATER TREATMENT PLANT

TABLE 3A-14
SUMMARY OF PHYSICAL-CHEMICAL (CHEMICAL FLOCCULATION) TREATMENT LIFECYCLE COSTS

Item	Cost
Capital	$ 97,000,000
Annual O&M	17,000,000
Present Worth	114,000,000

O&M = Operation and Maintenance

Source: 2020 Facilities Plan Project Files

3A.3.3 Technology: Jones Island Wastewater Treatment Plant Capacity

Introduction

The baseline treatment capacity at JIWWTP is 390 MGD (on a peak hour basis). This is composed of 330 MGD of secondary treatment by activated sludge and 60 MGD of blending. Flow is conveyed to the plant by gravity though the high and low level harbor siphons and is lifted into the plant by the influent screw pumps. The 2020 Baseline committed capacity of siphons/screw pump system is 330 MGD.[e] Consequently, the plant can not operate at full capacity unless some flow is pumped from the ISS to supplement the gravity flow to the plant.

The baseline capacity of the ISS pumps to JIWWTP is assumed to be 80 MGD. Because the plant capacity is 390 MGD and the maximum conveyance capacity of the siphons/screw pumps system is 320 MGD, the maximum ISS pumping capacity needed at the peak of the event is 60 MGD.

The wastewater treatment plant technologies identified in Chapter 2, *Technology/Indicator Analysis* were developed into combinations to be reviewed specifically at SSWWTP. Full secondary treatment can be achieved with either an activated sludge process (AS) or a physical-chemical (ballasted flocculation) process. Blending assumed flow was diverted directly from ISS pumping to disinfection. Disinfection is either by an ultra-violet method (UV) or a chlorine method (Cl). The physical-chemical (ballasted flocculation) process was identified in Chapter 2 as a technology that MMSD evaluated in a separate project from which data were provided.

Production Functions

In the development of the production functions, additional treatment capacity at JIWWTP is matched with additional ISS pump capacity; both increase in equal increments. The production functions extend to an additional capacity of 500 MG; that is, a total treatment capacity of 890 MGD and a maximum ISS pumping limit of 570 MGD.

[e] See Chapter 5, *Treatment Assessment – Future Condition* of *Treatment Report* for more details.

Additional treatment capacity is limited by available land area at JIWWTP. The estimated land area that could be used for additional treatment is 17.6 acres. This area is sufficient to add 100 MGD of treatment using the activated sludge method or 500 MGD of treatment using physical-chemical (ballasted flocculation) or blending methods.

Figure 3A-61 shows the overflow volumes removed by additional treatment capacity at JIWWTP. Both SSOs and CSOs are significantly reduced by additional treatment capacity at JIWWTP and by the additional ISS pumping capacity. With 500 MGD of additional capacity, almost all of the SSO volume is removed and half of the CSO volume is removed.

For comparison, this figure also shows the overflow volumes removed by additional treatment at SSWWTP. The SSO volume removed is essentially the same for both plants; except that the SSWWTP curve ends at 180 MGD and the JIWWTP curve extends to 500 MGD. The CSO volume is more effectively reduced by treatment at JIWWTP than at SSWWTP.

Figure 3A-62 shows the overflow events eliminated by additional capacity at JIWWTP. With 500 MGD of additional treatment capacity, only one SSO event remains and 40% of the CSO events are eliminated.

Cost Function

The cost functions for additional treatment capacity at JIWWTP are shown in Figure 3A-63. The cost values are the total present worth costs of the additional treatment and the additional ISS pumping. The cost functions are plotted against the design peak hourly flow of the additional treatment capacity. Compared to the costs at SSWWTP, treatment costs at JIWWTP are higher. For example, 100 MGD of additional treatment with full secondary treatment by the physical-chemical method and UV disinfection is $228 million at JIWWTP and $120 million at SSWWTP. The higher costs at JIWWTP are due to the additional ISS pumping and the land acquisition costs.

The difference in cost between the UV and the chlorine disinfection methods is negligible.

Cost Benefit Curves

The cost benefit curves for additional treatment at JIWWTP are shown in Figures 3A-64 to 3A-67 for the three treatment methods and the two disinfection methods.

Summary: Additional Treatment Capacity at Jones Island Wastewater Treatment Plant

Treatment at JIWWTP is a proven and effective technology to reduce SSOs and CSOs. Compared to SSWWTP, additional treatment capacity at JIWWTP is more expensive because the ISS pump capacity is increased along with the treatment capacity. The cost of land at JIWWTP further increases the overall cost of additional treatment capacity at JIWWTP.

Of the three treatment methods, the full secondary treatment by the physical-chemical method is the most cost effective option

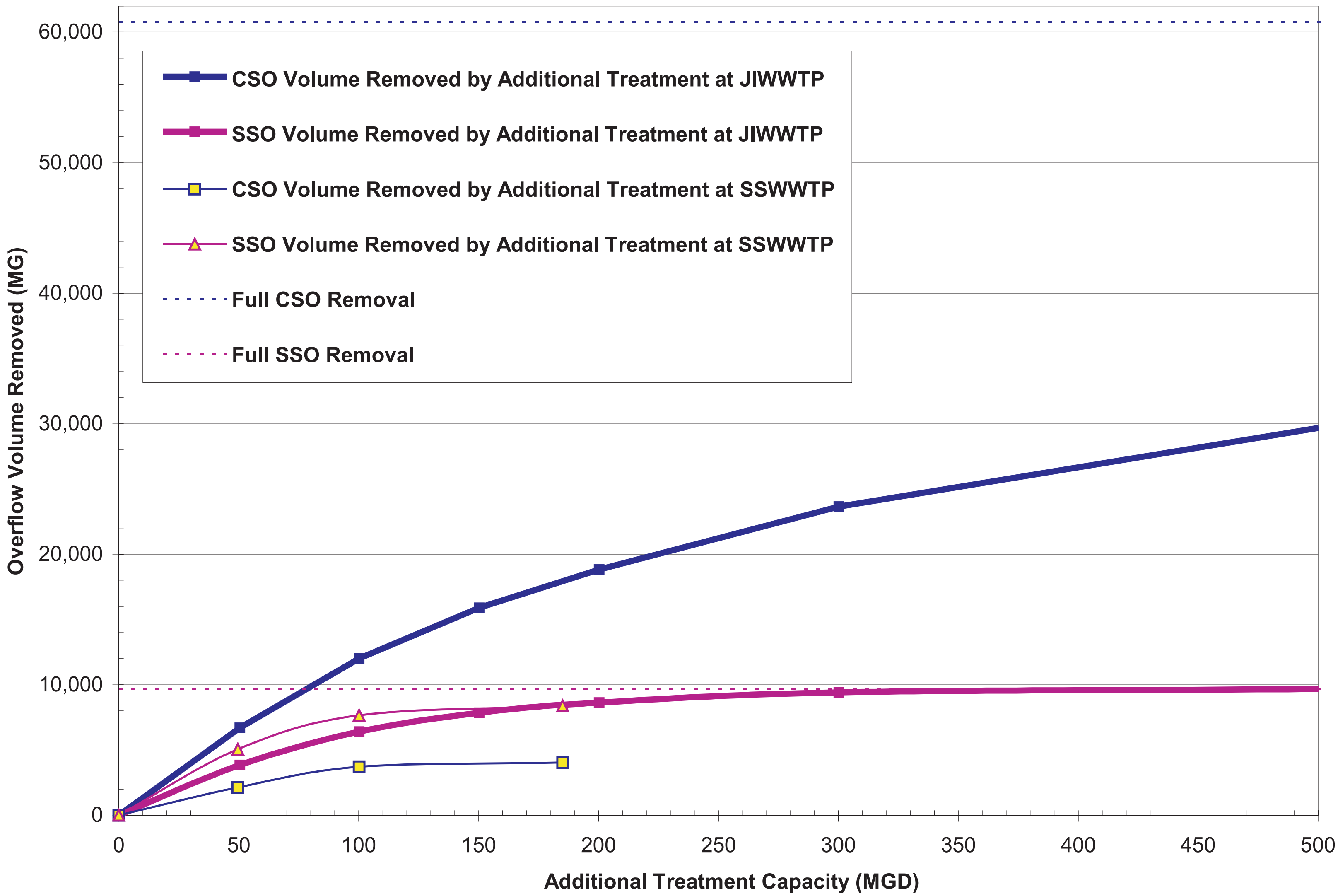

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FIGURE 3A-61
OVERFLOW VOLUME PRODUCTION FUNCTIONS: ADDITIONAL CAPACITY AT JIWWTP
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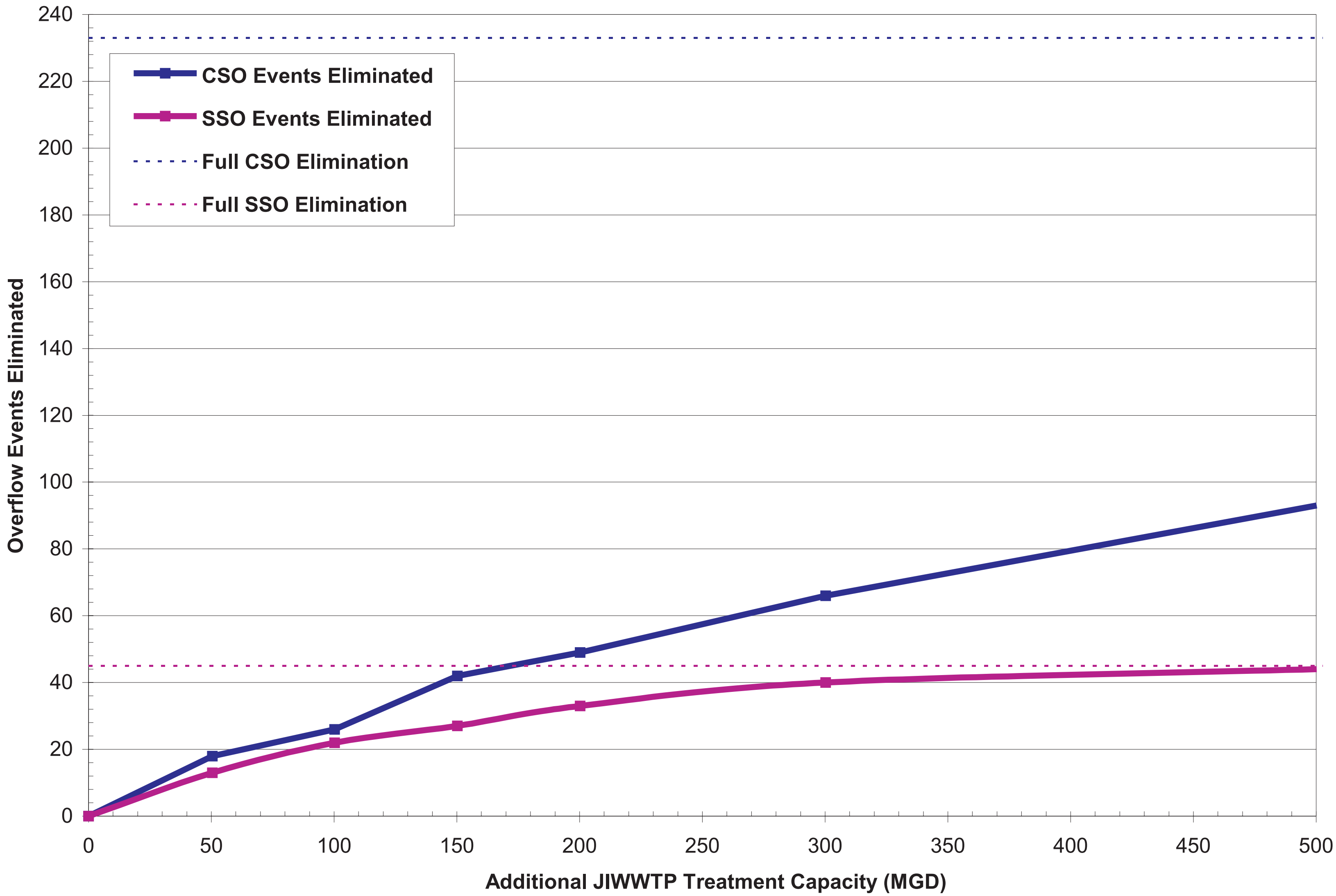

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FIGURE 3A-62
OVERFLOW EVENT PRODUCTION FUNCTIONS: ADDITIONAL CAPACITY AT JIWWTP
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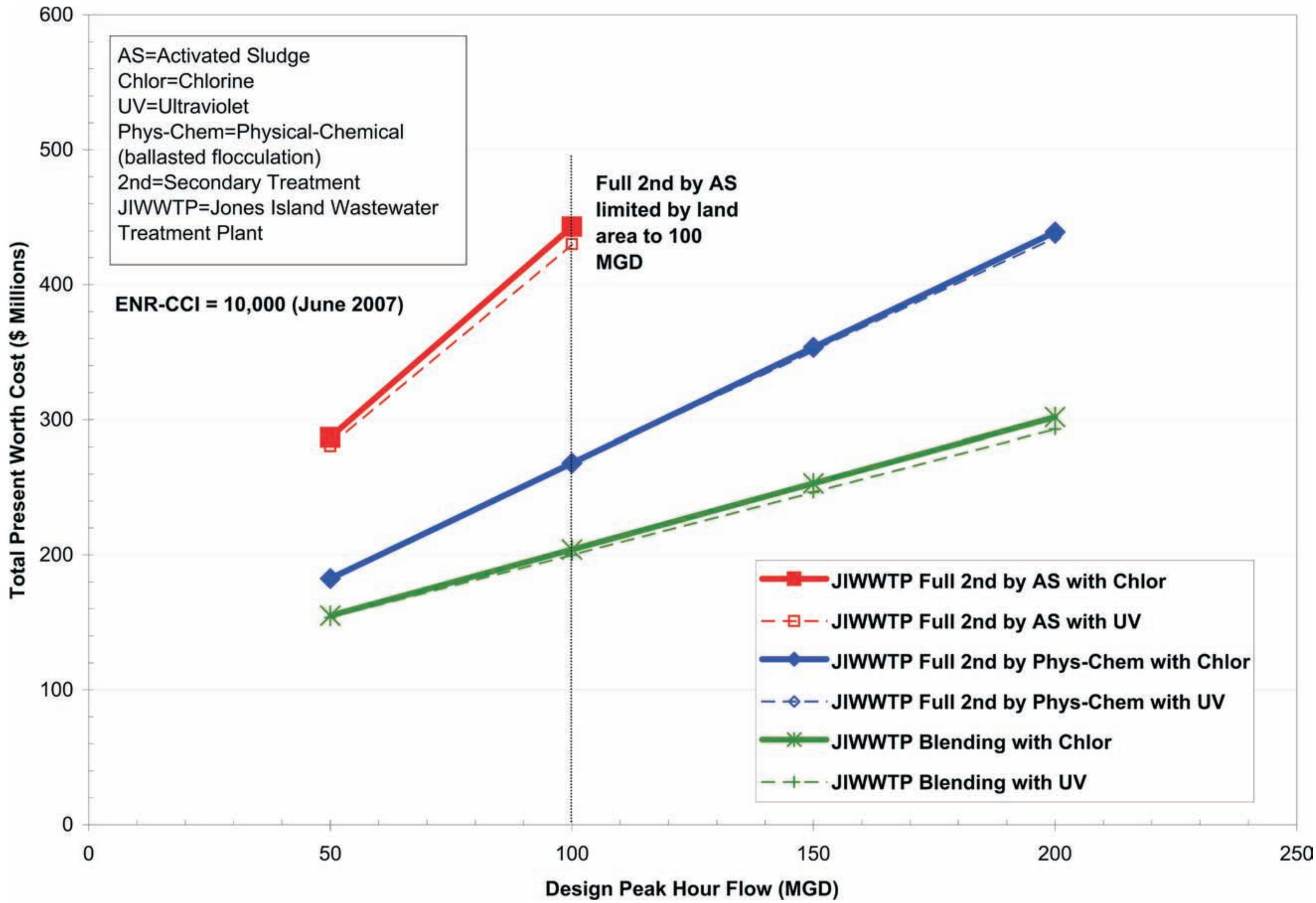

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FIGURE 3A-63
COST FUNCTIONS: ADDITIONAL JIWWTP CAPACITY
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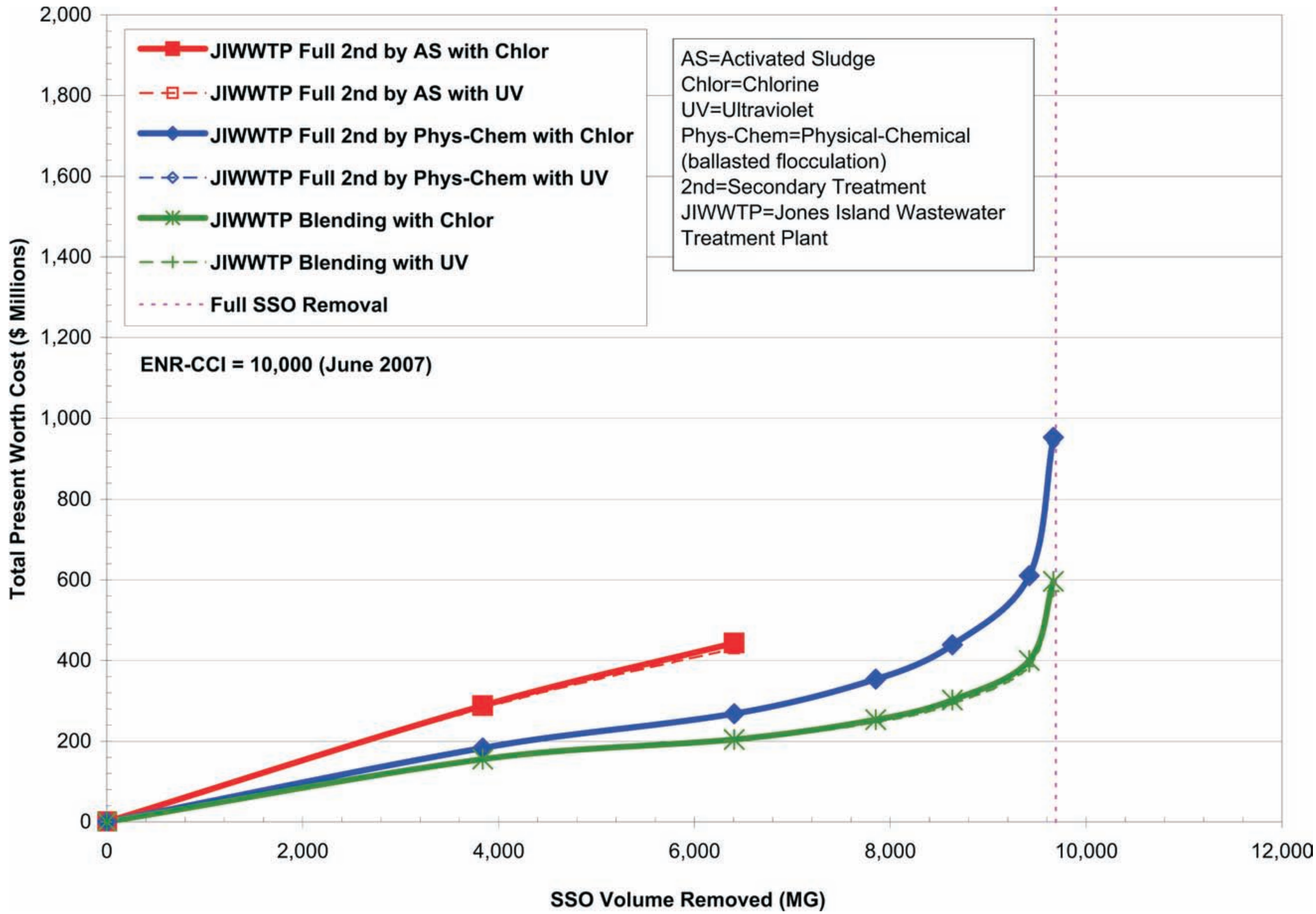

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FIGURE 3A-64

SSO VOLUME COST BENEFIT CURVES: ADDITIONAL TREATMENT AT JIWWTP

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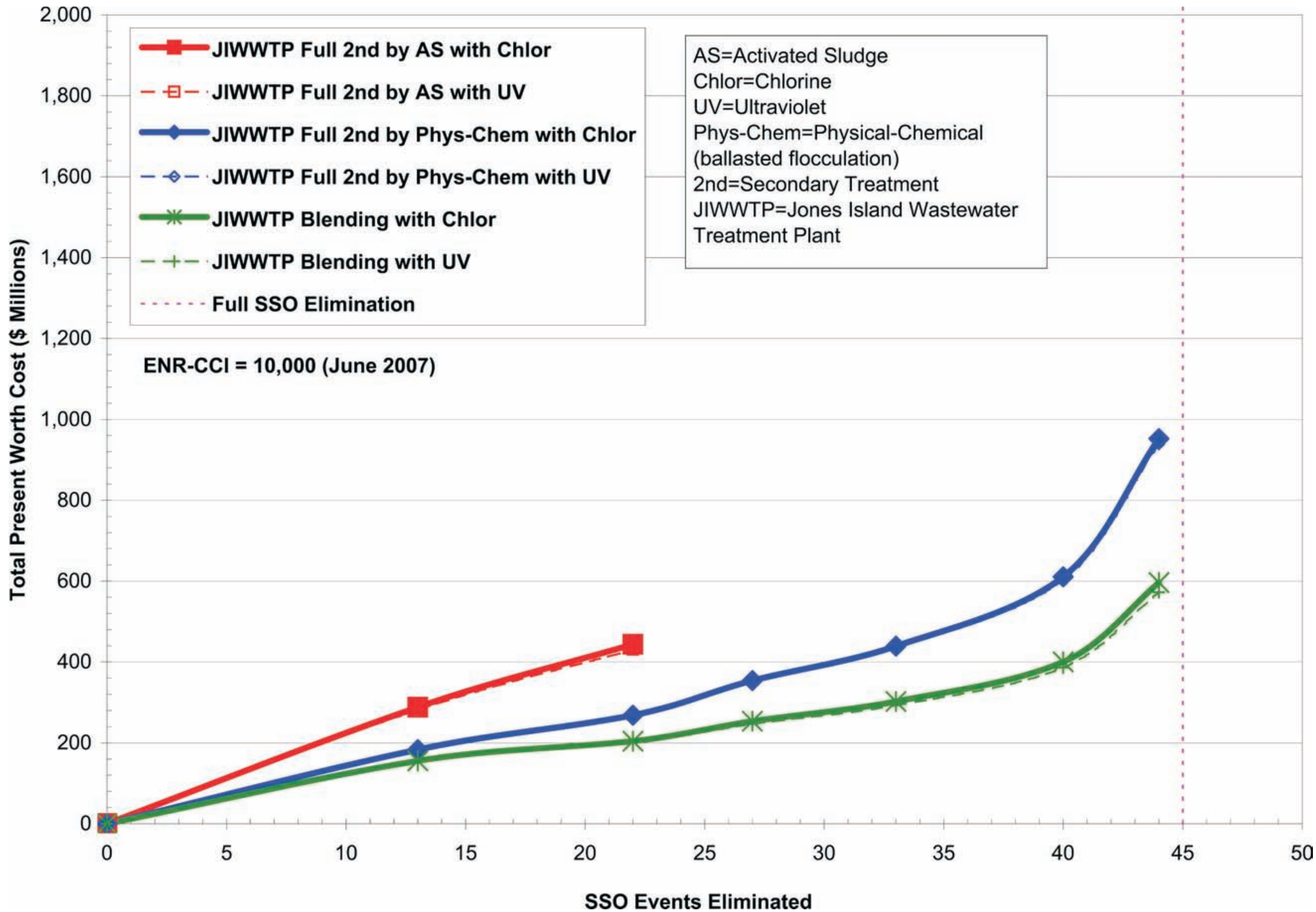

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FIGURE 3A-65

SSO EVENT COST BENEFIT CURVES: ADDITIONAL TREATMENT AT JIWWTP

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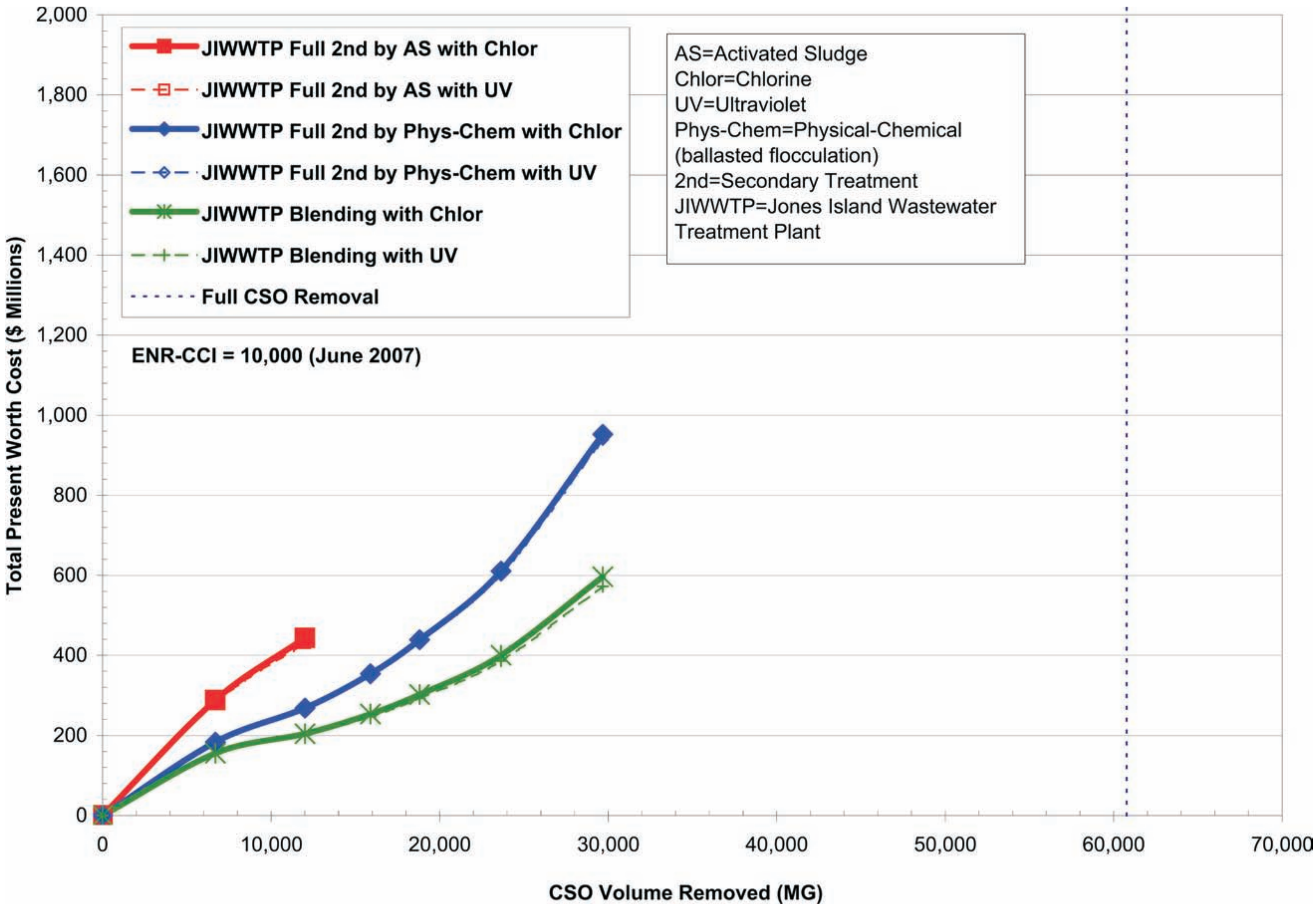

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FIGURE 3A-66

CSO VOLUME COST BENEFIT CURVES: ADDITIONAL TREATMENT AT JIWWTP

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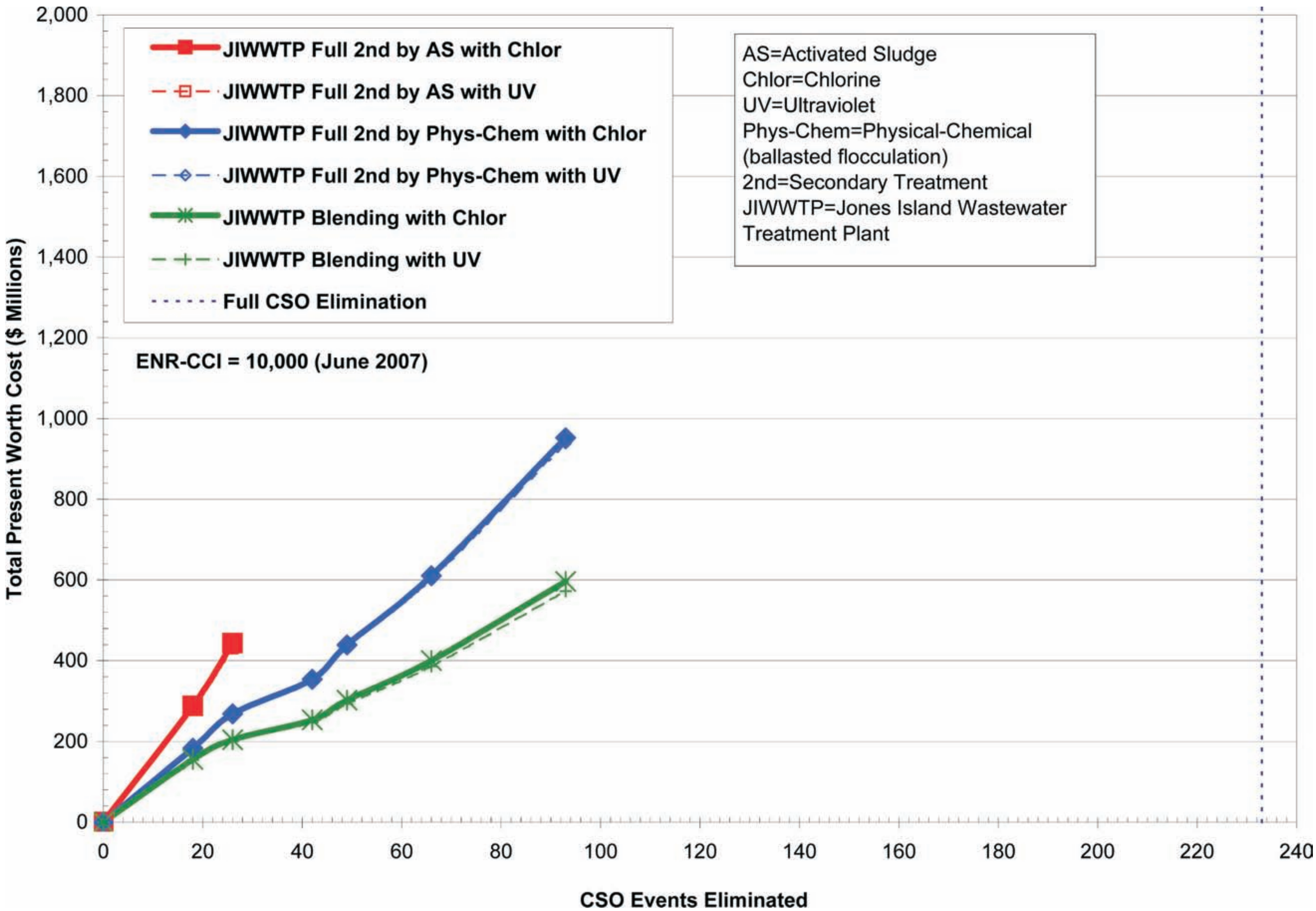

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FIGURE 3A-67
CSO EVENT COST BENEFIT CURVES: ADDITIONAL TREATMENT AT JIWWTP
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3A.3.4 Technology: Satellite Combined Sewer Treatment near CT-5/6

A conceptual satellite treatment plant near the CT-5/6 dropshaft could treat combined sewer flow before being discharged into the Menomonee River near 25th Street as a CSO. Very high flow rates in the CSO hydrographs are a substantial challenge to the feasibility of this technology. The flow in the CSSA is characterized by flashy, high intensity, short duration hydrographs. Some of the flow is stored in the ISS tunnel but after the tunnel gates close the excess flow becomes CSO discharge. A satellite treatment facility would need a very high capacity to treat the peak flow rate to prevent the CSO discharge.

Another problem with this technology is the limited time of operation. A satellite plant such as this would only operate during the peaks of large storms; most of the time the plant would not be operating. There are difficulties with maintaining and operating a plant that is normally dry and must rapidly come on-line for a few hours during the peak of a storm.

Model Configuration

The MACRO model is unable to model this concept; the model does not allow a satellite treatment facility to be added to the configuration. Nevertheless, the production function for this technology must be compatible with the other production functions that model the 64.5-year period of record. Therefore, the production functions for this technology are an interpretation of the MACRO results to infer the benefits of CSO satellite treatment near CT-5/6.

To support the MACRO model analysis, additional results from the MOUSE model were used. The MOUSE model generates hydrographs of the CSOs associated with each dropshaft in the CSSA. The hydrographs describe the CSO volume and peak flow rates for particular events. The production functions for a satellite treatment facility near CT-5/6 are derived from both the MOUSE model results and the MACRO model results.

MOUSE model results: CT-5/6 CSO hydrographs

Results from the MOUSE model were used to estimate the peak flow rate and the volume of the CSO hydrograph associated with the CT-5/6 area. There are five CSO outfall locations associated with CT-5/6. In the MOUSE model, the CSOs near CT-5/6 are represented by one hydrograph that is the sum of the flows at the five CSO locations.

Hydrographs for three events were used: March 1960, August 1986, and April 1999. Figure 3A-68 shows the CT-5/6 CSO hydrograph from MOUSE during the March 1960 event which had a high volume with a moderate intensity. The peak flow rate was 637 MGD and the volume was 135 MG. The duration of CSO discharge for this event was 24 hours.

Figure 3A-69 shows the hydrograph for the August 1986 event which was a very high intensity event. The peak flow rate was 3,090 MGD and the volume was 278 MG. The duration was 10 hours.

Figure 3A-70 shows the CT-5/6 CSO hydrograph for the April 1999 event which was a smaller event. The peak flow rate was 436 MGD and the volume was 47 MG. The duration was 6 hours.

These hydrographs do not represent historical flow events. They are simulated hydrographs that used the meteorological conditions at GMIA from these three dates to simulate the response of the system in the 2020 Baseline committed configuration. The hydrographs are used to define a relationship between the peak discharge rate that would have to be treated by a satellite treatment facility and the volume of the CSO hydrograph.

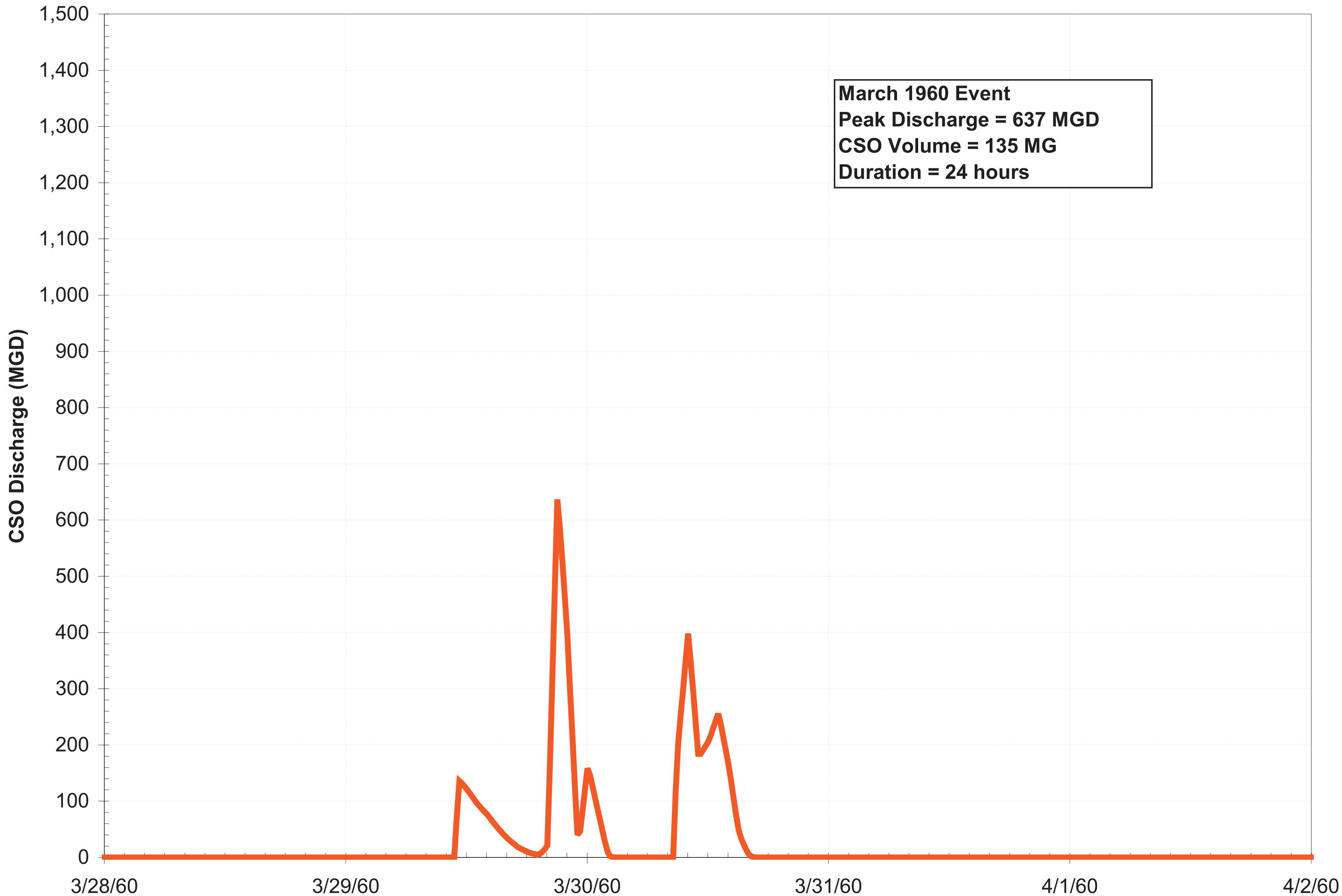

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FIGURE 3A-68
MOUSE MODEL RESULT, CSO HYDROGRAPH FOR CT-5/6, MARCH 1960
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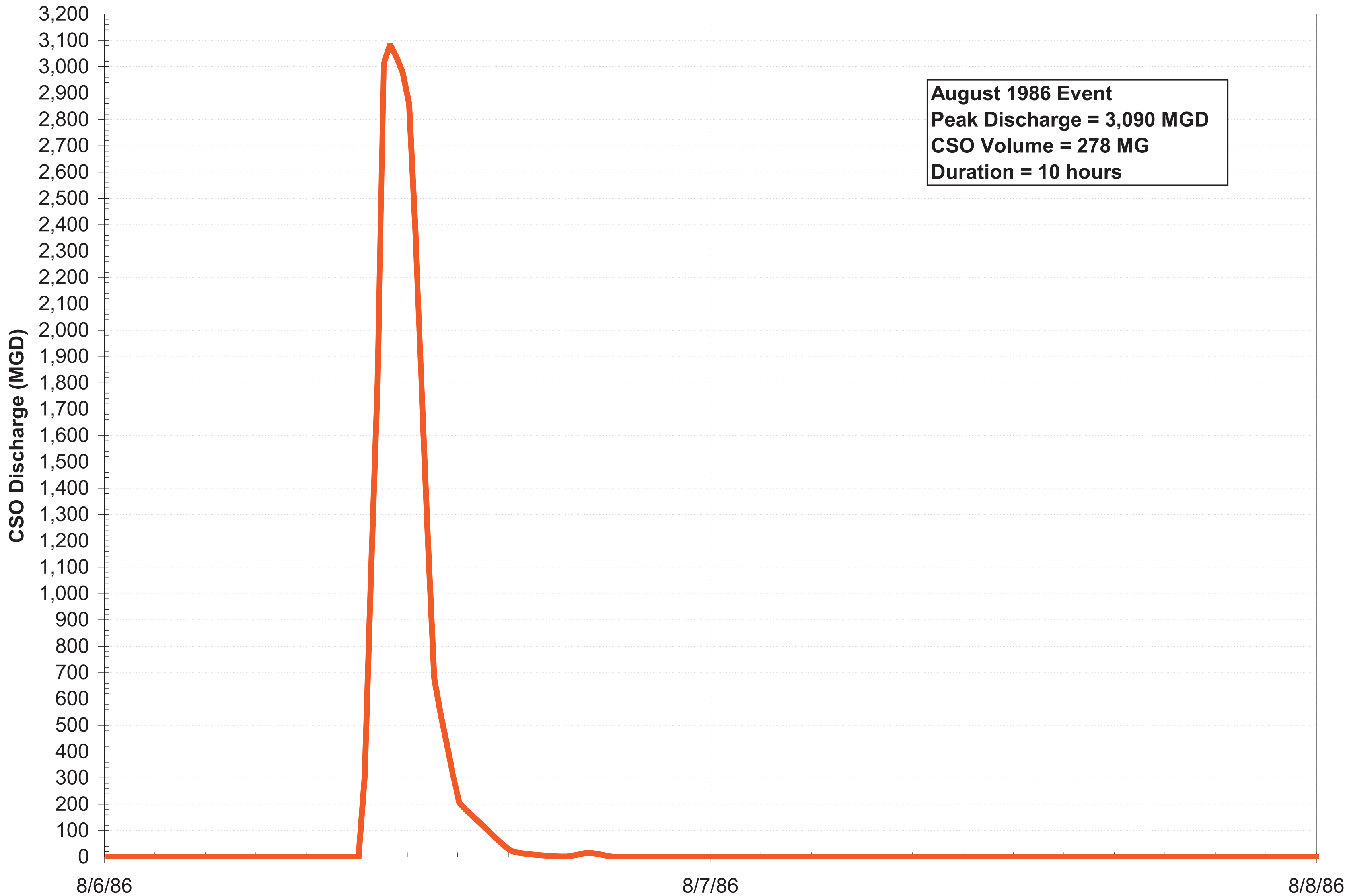

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FIGURE 3A-69

MOUSE MODEL RESULT, CSO HYDROGRAPH FOR CT-5/6, AUGUST 1986

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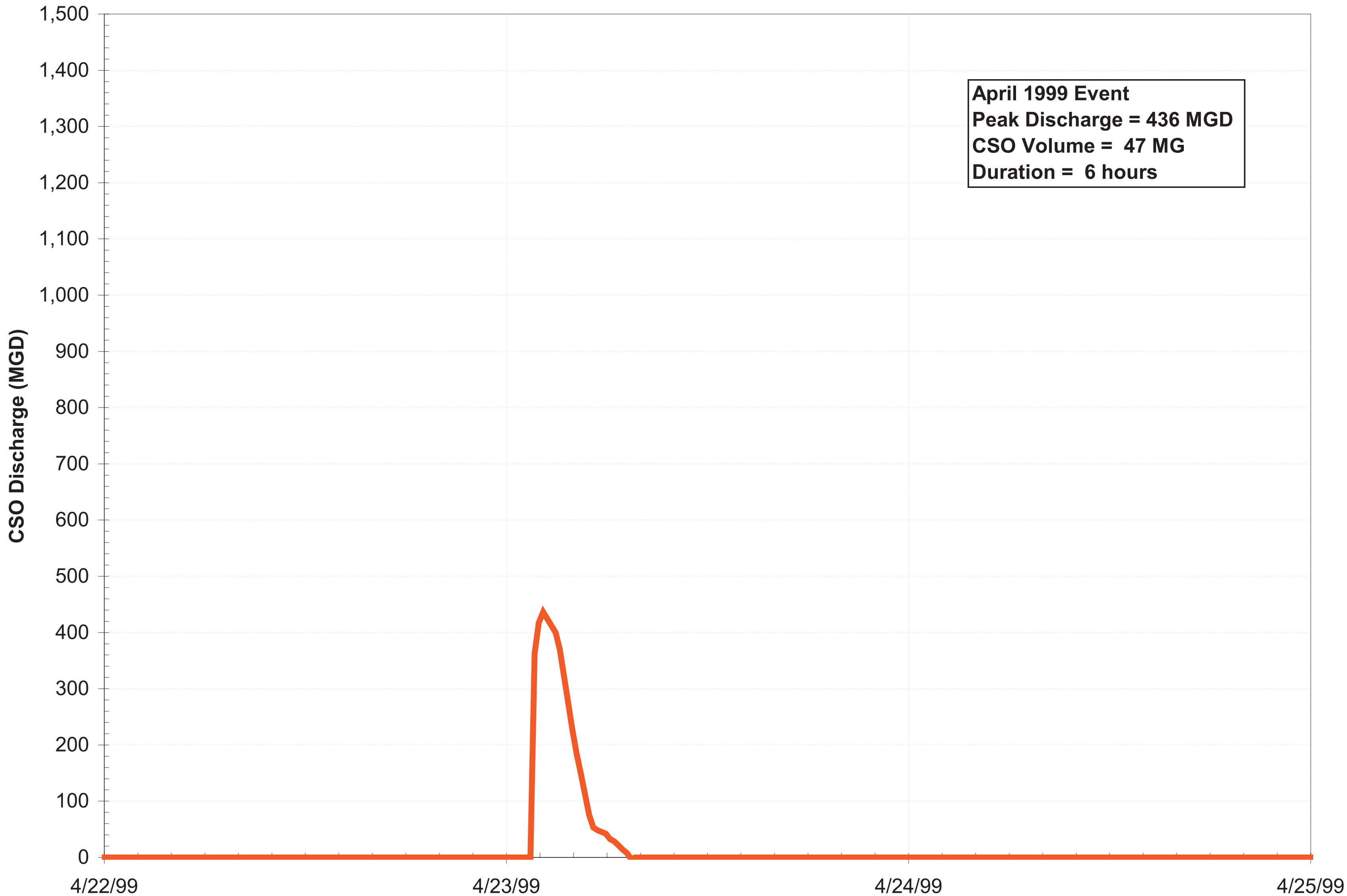

FIGURE 3A-70
MOUSE MODEL RESULT, CSO HYDROGRAPH FOR CT-5/6, APRIL 23, 1999

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MACRO Model Production Function for Deep Tunnel Storage as a Surrogate for Satellite Treatment at CT-5/6

MACRO can not model a satellite treatment facility. Therefore, the production functions for satellite treatment are an interpretation of the deep tunnel storage production functions. The interpretation is based on the MOUSE results that define the relationship between the peak CSO discharge rate and the volume of the CSO hydrograph. The key concept is that if a satellite treatment facility had sufficient capacity to treat the peak of the CSO flow, then the associated CSO volume would be eliminated. The deep tunnel production function shows the volume of CSO that can be removed by a certain amount of storage volume. It is assumed that the additional deep tunnel volume, in this case, is used exclusively for inflow from CT-5/6.

Figure 3A-71 is the production function for deep tunnel storage. Only the Case 2 production function curve for CSO volume removed is shown because in Case 2 the VRSSI is a constant value (150 MG). This case removes CSO volume efficiently because the additional tunnel volume is available for inflow from the CSSA.

Figure 3A-72 is a "zoom-in" window at a scale that provides better insight into the production function for the deep tunnel volume up to an additional 500 MG. On this curve the three events simulated in the MOUSE model are shown (March 1960, August 1986, and July 1997). These points are plotted based on the volume of the MOUSE CSO hydrographs for the three events.

The interpretation assumes that if the satellite treatment facility near CT-5/6 had a capacity of 436 MGD, then the April 1999 CSO event would be eliminated. This would be equivalent to the CSO eliminated by a deep tunnel volume of 47 MG, assuming that the additional deep tunnel volume was filled by flow from the CT-5/6 dropshaft.

For the March 1960 event, if the treatment facility had a capacity of 637 MGD, then it would eliminate the CSO of that event and it would be equivalent to the benefit of an additional 135 MG of deep tunnel storage.

For the August 1986 event, if the treatment facility had a capacity of 3,090 MGD, then it would eliminate the CSO of that event and it would be equivalent to the benefit of an additional 278 MG of deep tunnel storage.

Figure 3A-73 transforms the production function for deep tunnel storage into a production function for satellite treatment at CT-5/6. This production function assigns the peak satellite treatment rate to the horizontal axis in place of the additional storage volume. This production function is an estimate of the CSO volume removed during the 64.5 year period of record by satellite treatment capacity near CT-5/6.

If the CSO discharge at CT-5/6 were completely eliminated the total CSO volume would be reduced 70% from the baseline value. The remaining 30% of the CSO volume is due to the other CSO sites.

These treatment capacities required for the satellite WWTP are very large. To fully treat the largest hydrograph at CT-5/6, the satellite treatment facility would need a peak capacity of 3,090 MGD. A treatment facility this large is unrealistic. Even to treat the CSO from the April 1999 event, the facility would need a capacity of 436 MGD.

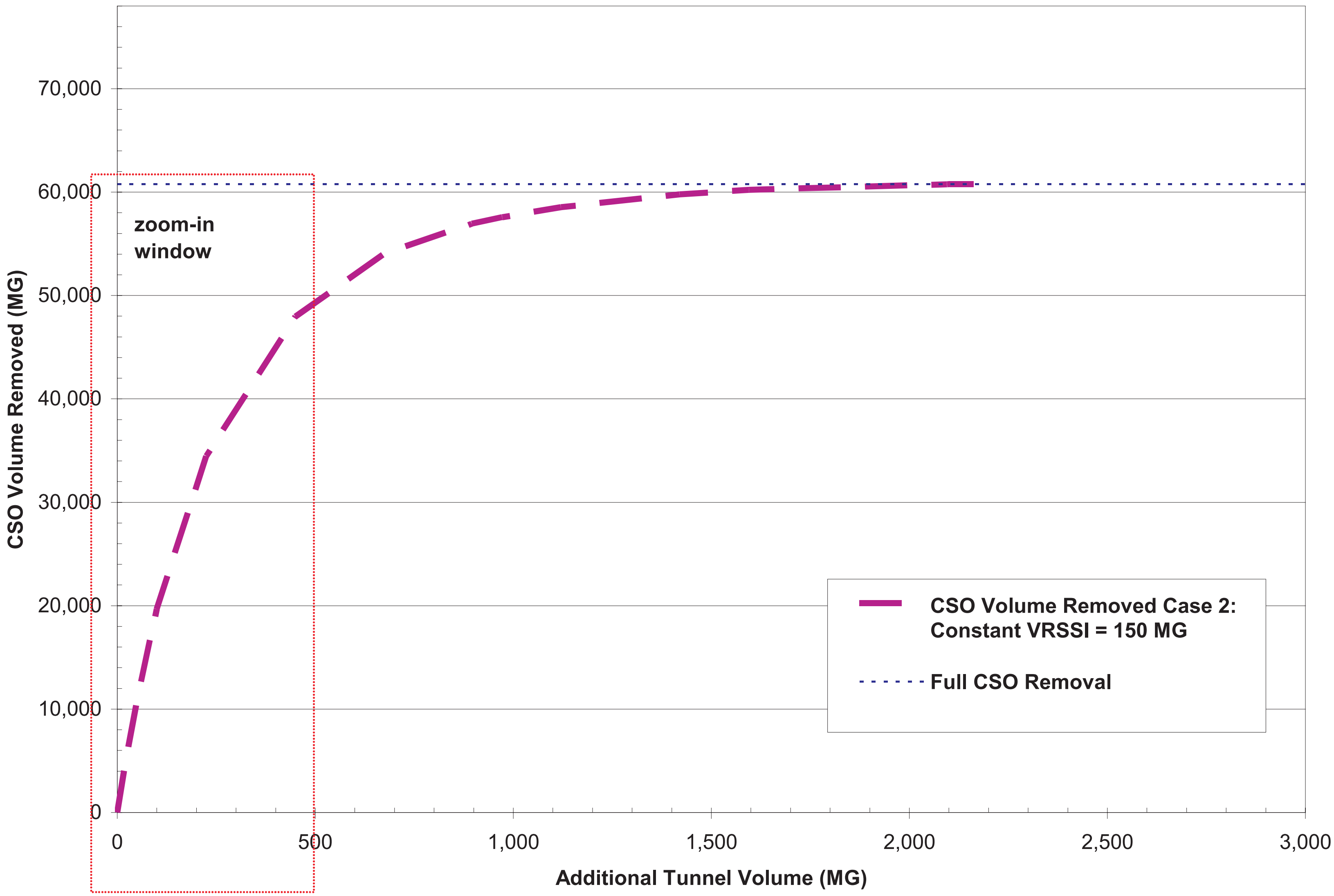

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FIGURE 3A-71

CSO VOLUME PRODUCTION FUNCTION: SATELLITE TREATMENT USING ADDITIONAL DEEP TUNNEL STORAGE AS SURROGATE

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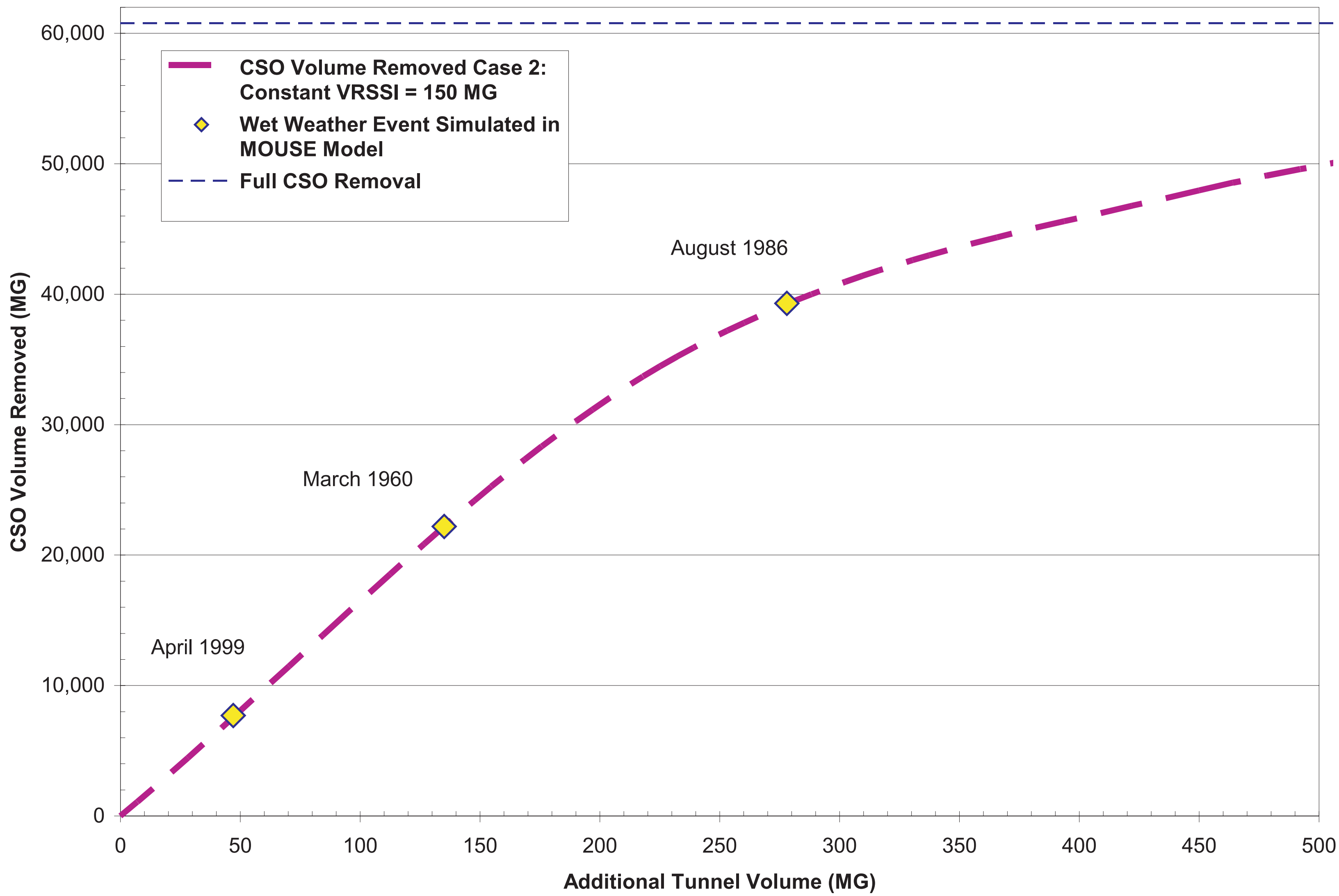

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FIGURE 3A-72

CSO VOLUME PRODUCTION FUNCTION: SATELLITE TREATMENT USING ADDITIONAL DEEP TUNNEL STORAGE AS SURROGATE (ZOOM-IN WINDOW)

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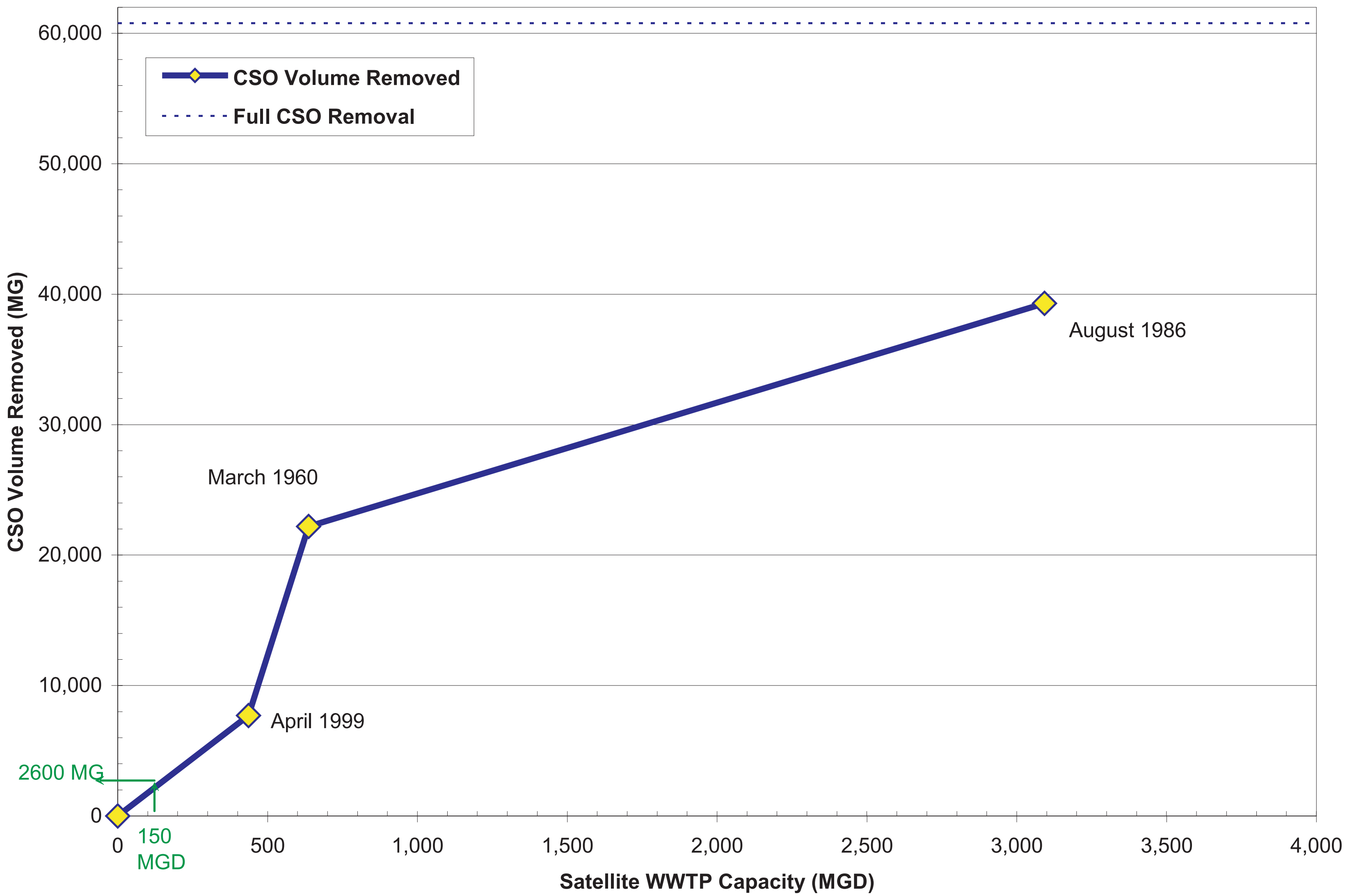

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FIGURE 3A-73
CSO VOLUME PRODUCTION FUNCTION:
AUXILIARY WWTP AT CT-5/6
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A facility with a smaller capacity, such as 150 MGD, would produce an insignificant benefit for CSO reduction. For example, using the production function in Figure 3A-72, 150 MGD of satellite treatment capacity could remove 2,600 MG during the period of record; which is only 4% of the total CSO volume.

Comparison to Treatment at Jones Island Wastewater Treatment Plant

The effectiveness of satellite treatment near CT-5/6 is small. An equal amount of additional treatment capacity at JIWWTP is much more effective.

Figure 3A-74 shows the production function for additional treatment capacity at JIWWTP. An additional 150 MGD of capacity at JIWWTP can remove 16,000 MG during the period of record (26% of the total CSO volume). The efficiency of treatment at JIWWTP is six times better than satellite treatment near CT-5/6. In addition, the additional capacity at JIWWTP has benefits for SSOs as well as CSOs. Therefore, additional treatment at JIWWTP is a significantly more cost effective alternative to a satellite facility near CT-5/6.

Summary: Satellite Wastewater Treatment Plant near CT-5/6

A satellite WWTP near CT-5/6 is not a feasible technology for the following reasons:

- The treatment capacity needed to treat CSOs must accommodate a very high flow rate
- The time of operation is very brief, making operation difficult and inefficient
- More cost-efficient and comprehensive benefits can be achieved by additional treatment capacity at JIWWTP

To fully remove CSOs at a satellite facility near CT-5/6, the peak treatment capacity would need to be 3,090 MGD. This is a very large value. Furthermore, the plant would need to operate on an intermittent basis. During a large CSO event the facility might only operate for a few hours. The plant would be idle for extended periods of time.

Additional treatment at JIWWTP is much more effective than satellite treatment. Both SSOs and CSOs are reduced by additional capacity at JIWWTP and the benefits are not localized to one part of the service area. Additional capacity at JIWWTP would be used more frequently and for longer durations because it can also be used for rapid dewatering of the ISS.

3A.4 Technology: Infiltration and Inflow Reduction

3A.4.1 Overview of Approach

Infiltration and inflow can cause the wastewater flow rates in the collection system to rise significantly during wet weather. Excessive peak flows and greater wastewater volumes result in SSOs when the conveyance, storage, and treatment capacities of the system are exceeded.

Rehabilitation of the collection systems to reduce the I/I component of the wastewater flow is a potential technology to improve the level of protection against SSOs. This analysis used a calibrated sewershed flow model to characterize the current I/I response at the sewershed scale. Conceptual reductions in I/I were then analyzed to predict the system-wide cost benefit relationship of I/I reduction efforts.

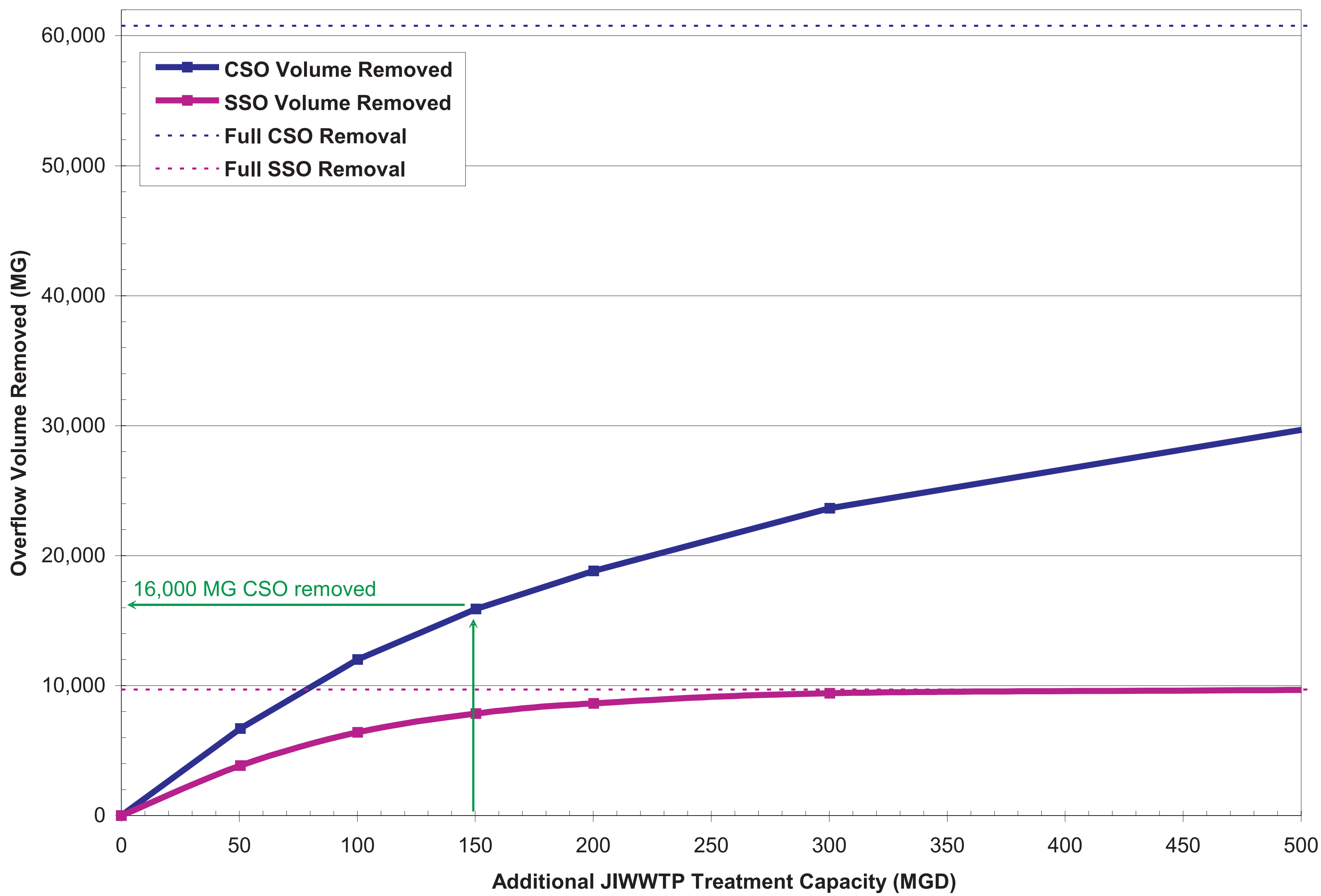

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FIGURE 3A-74
OVERFLOW VOLUME PRODUCTION FUNCTIONS: ADDITIONAL JIWWTP CAPACITY
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The I/I reduction analysis was applied on the sewershed scale; this means that I/I rates and potential reductions in I/I were estimated for individual sewersheds. No analysis was performed for areas smaller than a sewershed. Each sewershed contains sewer pipes and other collection facilities that are potential sources of I/I. These pipes and facilities may be owned by MMSD (interceptor sewers), local communities (sewer mains and trunk sewers), or private property owners (laterals). The analysis did not partition the costs to the various ownership domains; instead, the cost was estimated for a sewershed as a whole.

Because the costs were estimated at the sewershed scale, the method did not assume a particular type of I/I reduction technology. Common I/I reduction technologies, identified in Chapter 2, *Technology/Indicator Analysis*, generally include rehabilitating private property sewer laterals, disconnecting private property drains, replacing or relining sewer mains, repairing manholes, and eliminating stormwater cross-connections with the sanitary sewer system. This analysis did not assume the application of specific technologies as the data required to take this approach were not available in the MMSD service area. Even if data were available on the number of such potential sources of I/I, it would be difficult to predict the amount of I/I reduction each of them represents. Instead, a performance-based approach was used that focused on the outcome of the I/I reduction efforts, which was a function of the investment made in I/I reduction. The performance-based approach used a unit cost function, which was derived from several demonstration projects and real-life rehabilitation projects that were carefully monitored and evaluated for effectiveness and cost.

Appendix 9A, *Infiltration and Inflow Reduction* of the *Conveyance Report* explains the I/I reduction technology methodology in further detail, including the assumptions and the development of the method. That appendix also shows how I/I reduction analysis was applied to determine the cost effective level of system-wide I/I reduction. This section presents the I/I reduction production and cost functions used to create the cost benefit curve.

3A.4.2 Baseline Infiltration and Inflow in Sewersheds (Pre-Rehabilitation Rates)

Peak hour I/I rates for all sewersheds during a 5-year event were compared using the flow per unit area, with units of gallons per acre per day (gpad). Figure 3A-75 shows the range of sewershed I/I rates. A few sewersheds had I/I rates over 30,000 gpad but most sewersheds were less than 10,000 gpad (the median was approximately 6,500 gpad). The overall average I/I rate (based on the sum of the peak I/I flows divided by the sum of the separate area served) was 4,500 gpad.

Figure 3A-76 shows the relative distribution of the source of I/I flow as a percent of the separate sewer service area served. Half of the peak wet weather flow was generated in 15% of the separate sewer area. The other half of the peak wet weather flow came from the remaining 85% of area. This implies that I/I work must be focused on those sewersheds that make the largest relative contribution to the I/I flow.

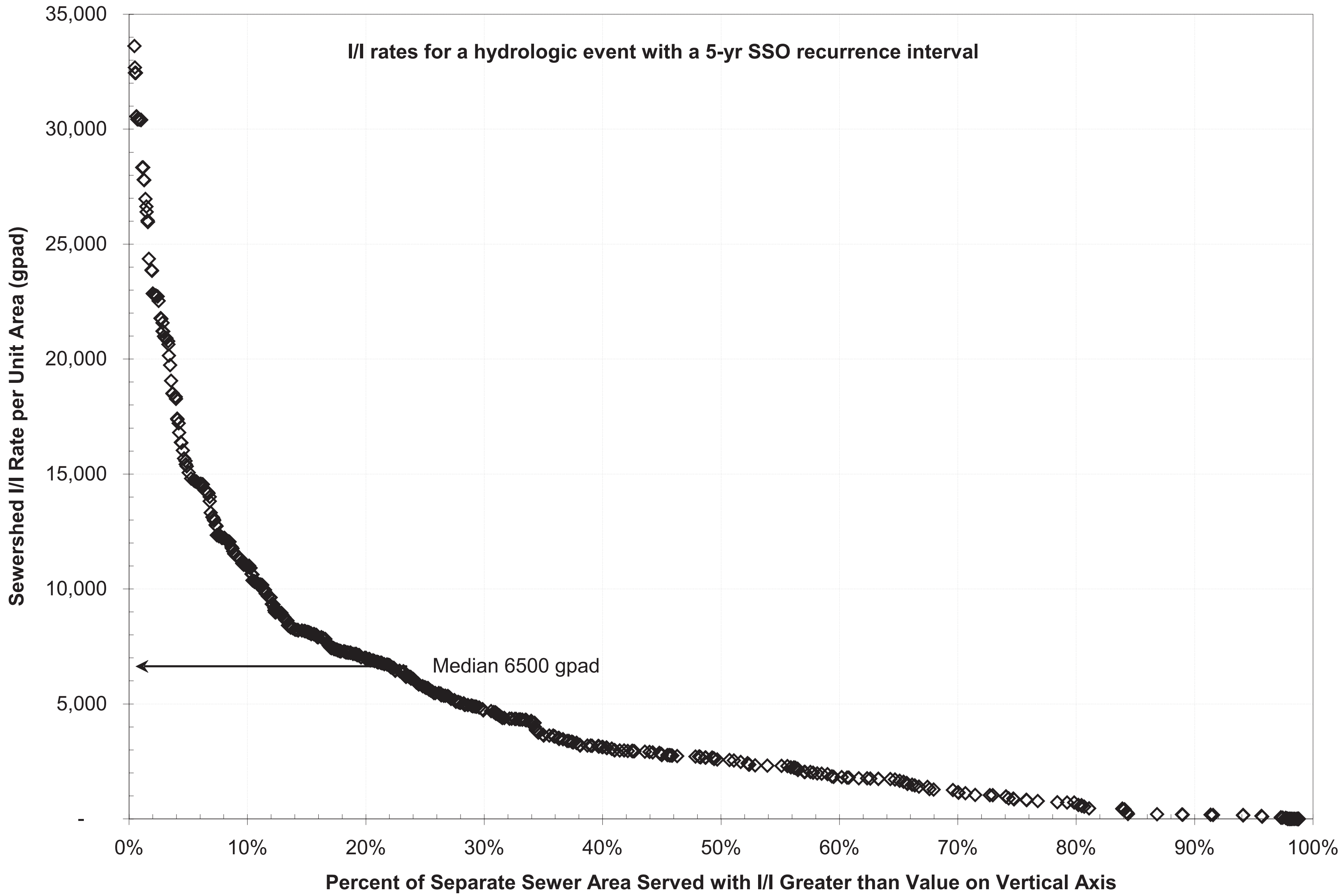

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FIGURE 3A-75

DISTRIBUTION OF SEWERSHED I/I RATES PER UNIT AREA

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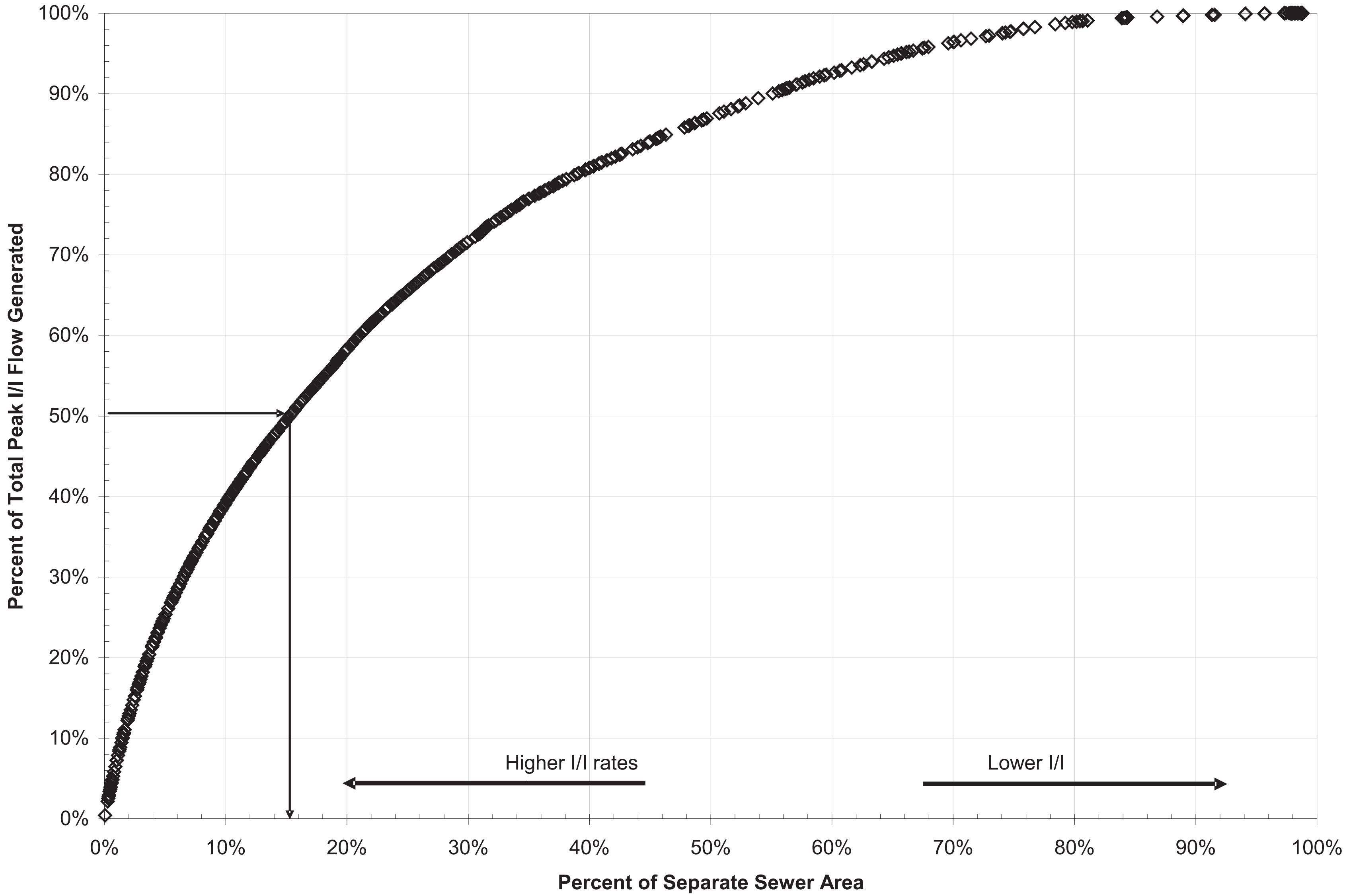

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FIGURE 3A-76

RELATIVE DISTRIBUTION OF I/I PEAK FLOW

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3A.4.3 Infiltration and Inflow Reduction – Performance Based Unit Cost Function

A key component to this approach is the unit cost curve used to determine the investment required to achieve each target I/I level for each sewershed. Development of the curve reflects results from over twenty individual I/I reduction projects that were analyzed for cost and I/I reduction in a fashion compatible with a Water Environment Research Foundation (WERF) protocol.(14) This protocol detailed what specific information should be provided when documenting an I/I reduction project and how to perform an analysis of I/I reduction effectiveness. By using proper documentation and protocols, the results of various I/I reduction projects can be examined equally and used by others who are preparing to do similar work.

Three specific sources were used for developing the I/I project data set:

- MMSD I/I Reduction Demonstration Projects(15)
- King County I/I Reduction Pilot Projects(16)
- WERF I/I Reduction Projects(17)

For purposes of comparing results from these studies, the pre-rehabilitation 5-year wastewater recurrence interval event flow was chosen and plotted in Figure 3A-77 with the project unit cost in terms of dollars spent per gpd removed. The unit cost curve presented in Figure 3A-77 represents a power curve that fits an upper trend of data, and is therefore a conservative fit. This approach was chosen because it conformed to a basic premise from the pilot project reports mentioned above – that the unit cost of I/I reduction decreases for the leakier conditions. Also, as pre-rehabilitation I/I rates decrease, the unit cost of I/I reduction increases.

3A.4.4 Infiltration and Inflow Reduction – Calculation Method

The first level of I/I reduction identified all sewersheds with I/I rates greater than 30,000 gpad; at this level, 18 sewersheds (representing 1% of the SSSA) were effected. Infiltration and inflow rehabilitation efforts are assumed to successfully reduce the I/I rates in these sewersheds to 30,000 gpad while the I/I rates in the remainder of the separate sewer area were unchanged.

The second level reduced I/I rates to 25,000 gpad. This level assumes additional work in the 18 sewersheds identified in the first level to bring those rates from 30,000 gpad to 25,000 gpad and rehabilitation in 10 other sewersheds that had initial rates between 25,000 and 30,000 gpad. The 28 sewersheds affected at this level represent 2% of the SSSA.

Figure 3A-78 illustrates how the process was repeated for I/I levels of 20,000, 15,000, and 10,000 gpad. Table 3A-15 summarizes the number of sewersheds, percent of area, and the percent reduction in system-wide peak flow at each I/I reduction level.

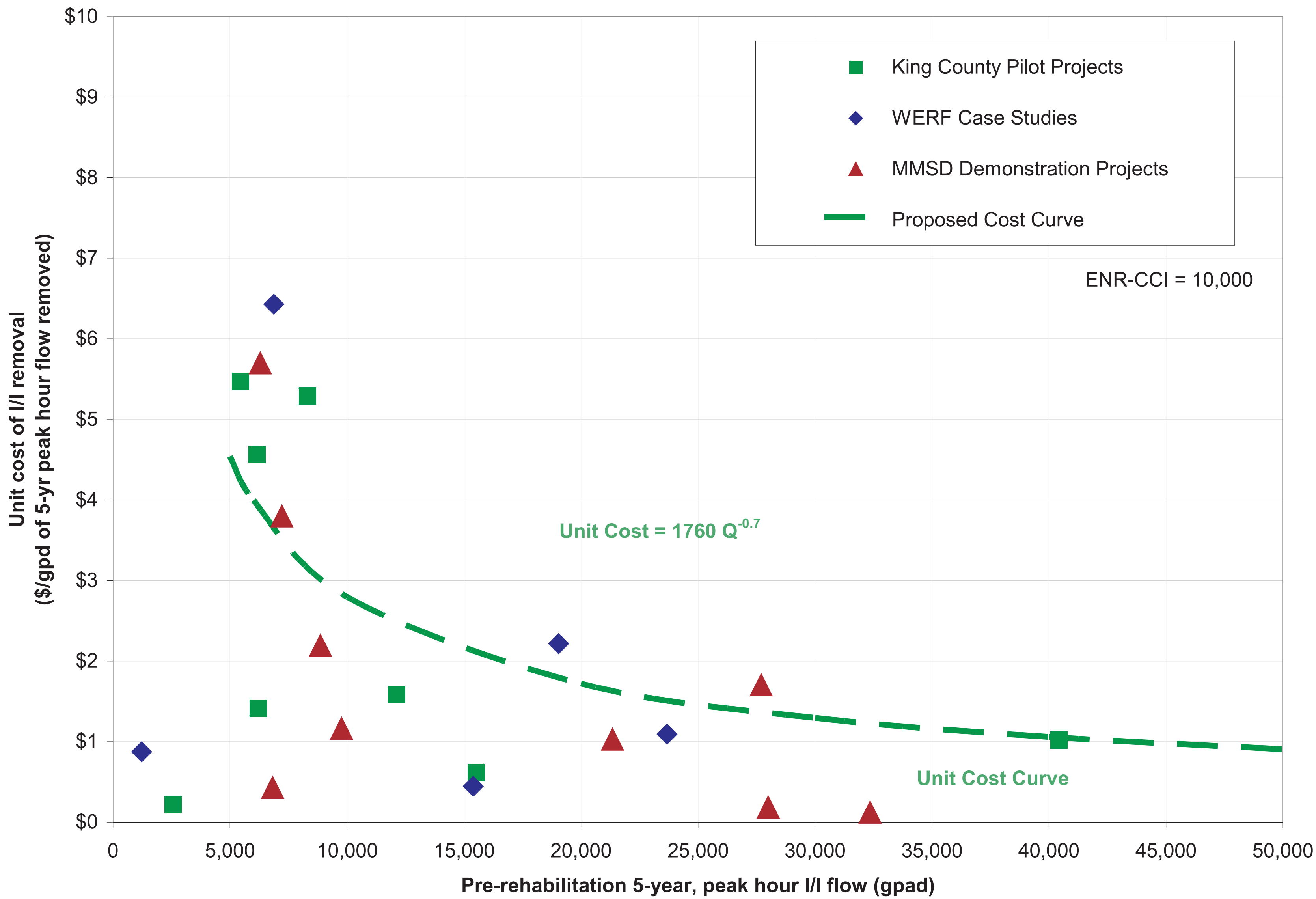

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FIGURE 3A-77
PERFORMANCE-BASED UNIT COST: I/I REMOVAL
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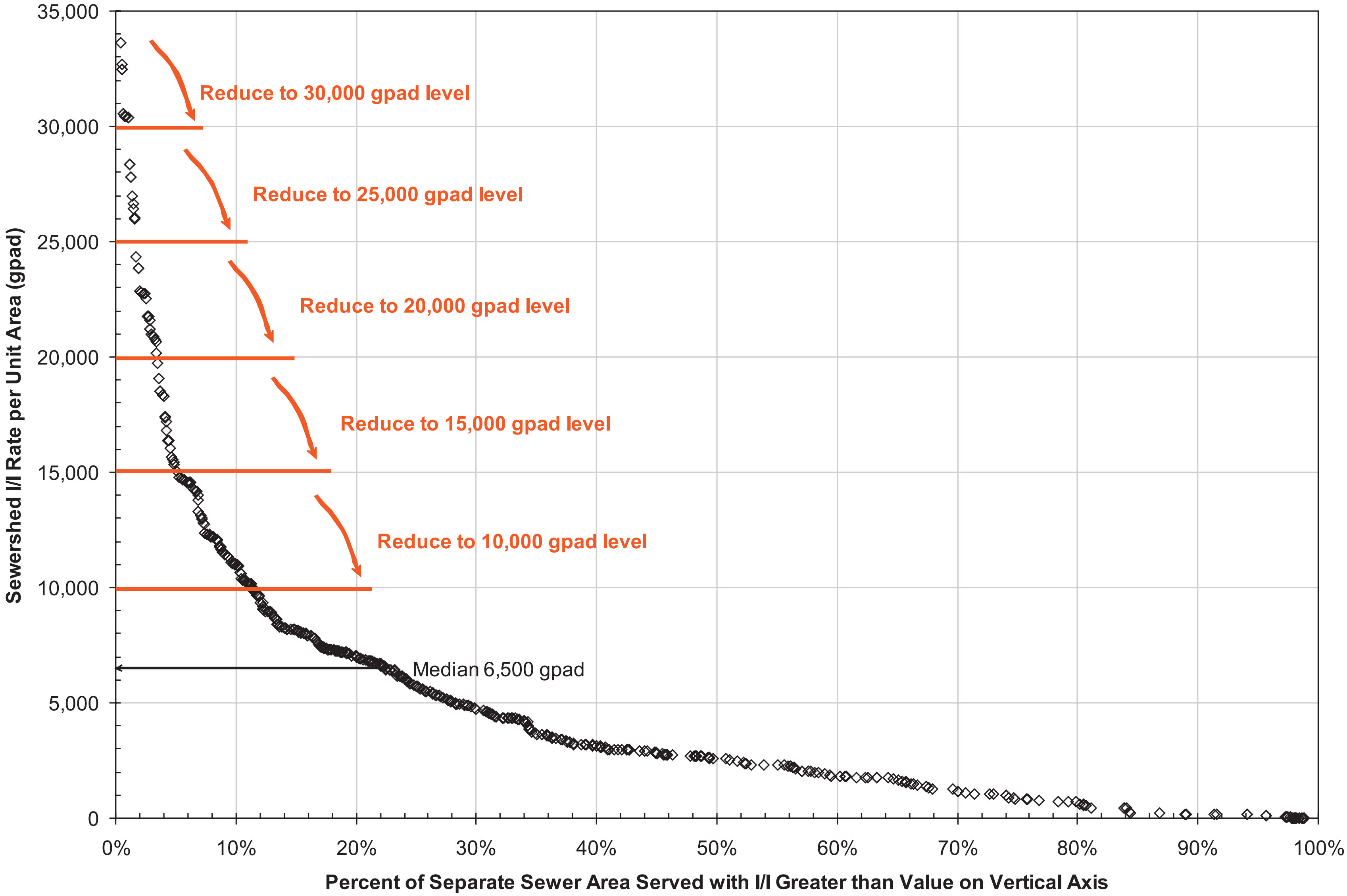

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FIGURE 3A-78
PROGRESSIVE LEVELS OF I/I REDUCTION
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TABLE 3A-15

NUMBER OF SEWERSHEDS AFFECTED BY I/I REDUCTION LEVELS

Infiltration and Inflow Reduction Level	Number of Sewersheds Affected by I/I Rehabilitation	% of Separate Sewer Service Area	% Reduction in Peak I/I Flow
30,000 gpad	18	1	1
25,000 gpad	28	2	2
20,000 gpad	49	3	5
15,000 gpad	68	5	10
10,000 gpad	140	11	19
7500 gpad	201	17	26
6000 gpad	279	24	34
5000 gpad	310	28	39

gpad = Gallons per Acre per Day
I/I = Infiltration and Inflow

3A.4.5 Cost Function

The unit cost of the I/I reduction is based on the pre-rehabilitation I/I rate. When progressing through the I/I levels, the I/I level resulting from the previous step is used as the pre-rehabilitation rate for the next level. The sum of the cost for all affected sewersheds is the incremental construction cost for that I/I reduction increment, or the cost of I/I reduction to get from one I/I level to the next.

The cumulative construction cost is the sum of the incremental costs to reach a desired I/I reduction goal. The construction cost includes a 25% contingency.

The capital cost includes 35% of the construction cost for technical services (engineering and administration). Figure 3A-79 is the cumulative capital cost as a function of the percent reduction in system-wide peak I/I flow as these I/I reduction goals are achieved.

3A.4.6 Production Functions

Production functions for I/I reduction are based on MACRO simulations that determined the number of events eliminated and the volume of overflow removed for each I/I reduction level. The production functions compare the percent change in overflow events or volume to the percent reduction in the system-wide peak I/I flow. Figure 3A-80 shows the production functions for SSO volume removed by I/I reduction. Figure 3A-81 shows the production functions for SSO events eliminated. I/I reduction is a technology that has a primary benefit for reducing SSOs with secondary benefits in CSO reduction that occur under special circumstances.

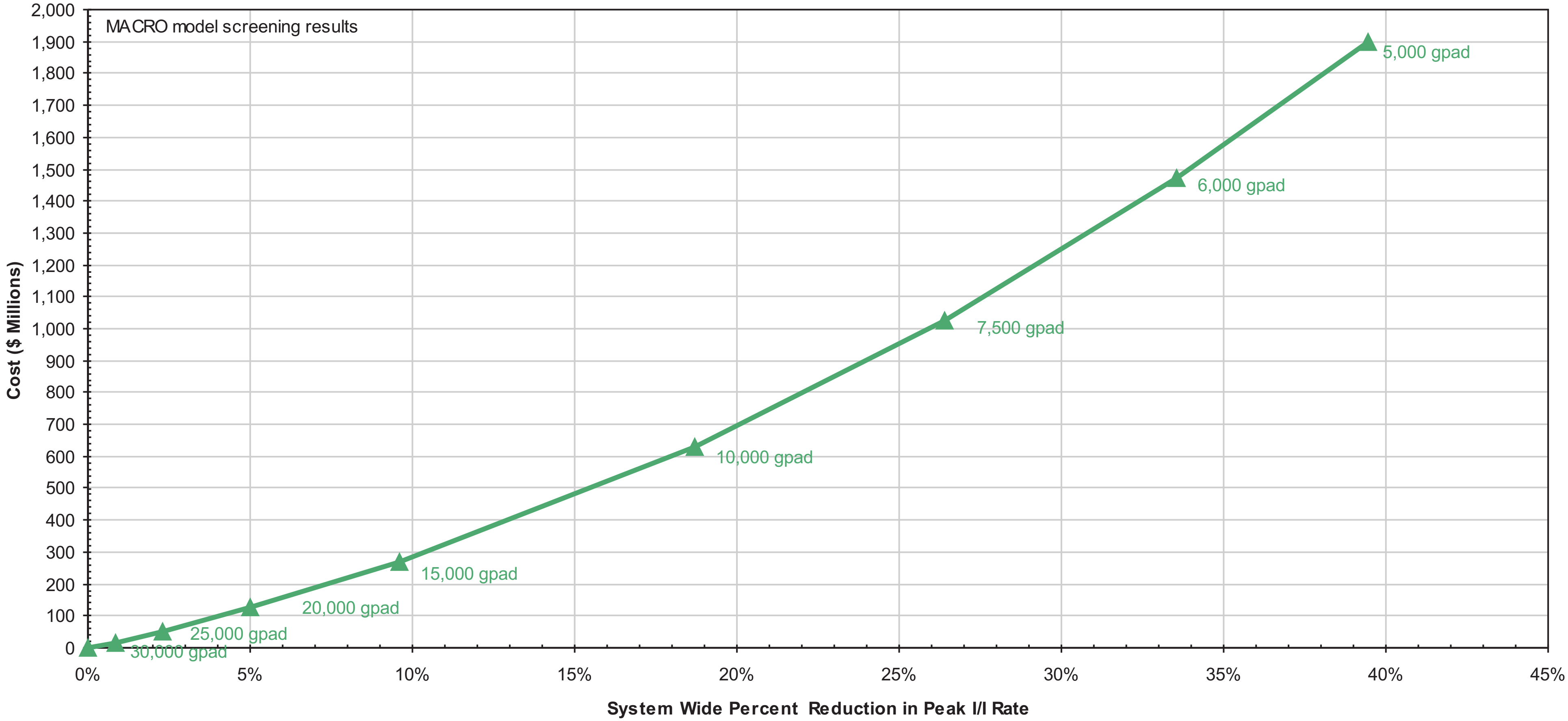

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FIGURE 3A-79
COST FUNCTION:
I/I REDUCTION
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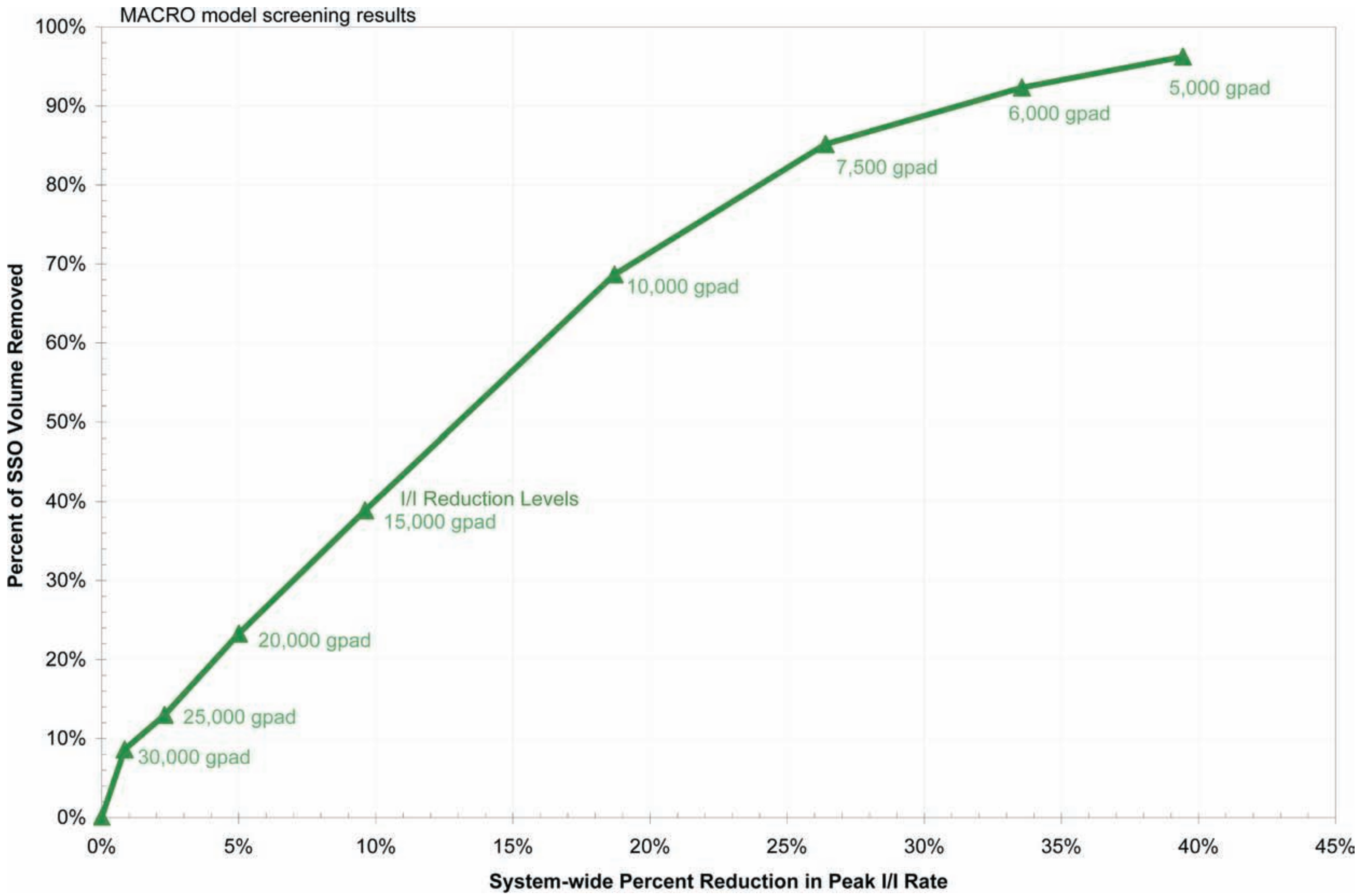

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FIGURE 3A-80
SSO VOLUME PRODUCTION FUNCTION: I/I REDUCTION
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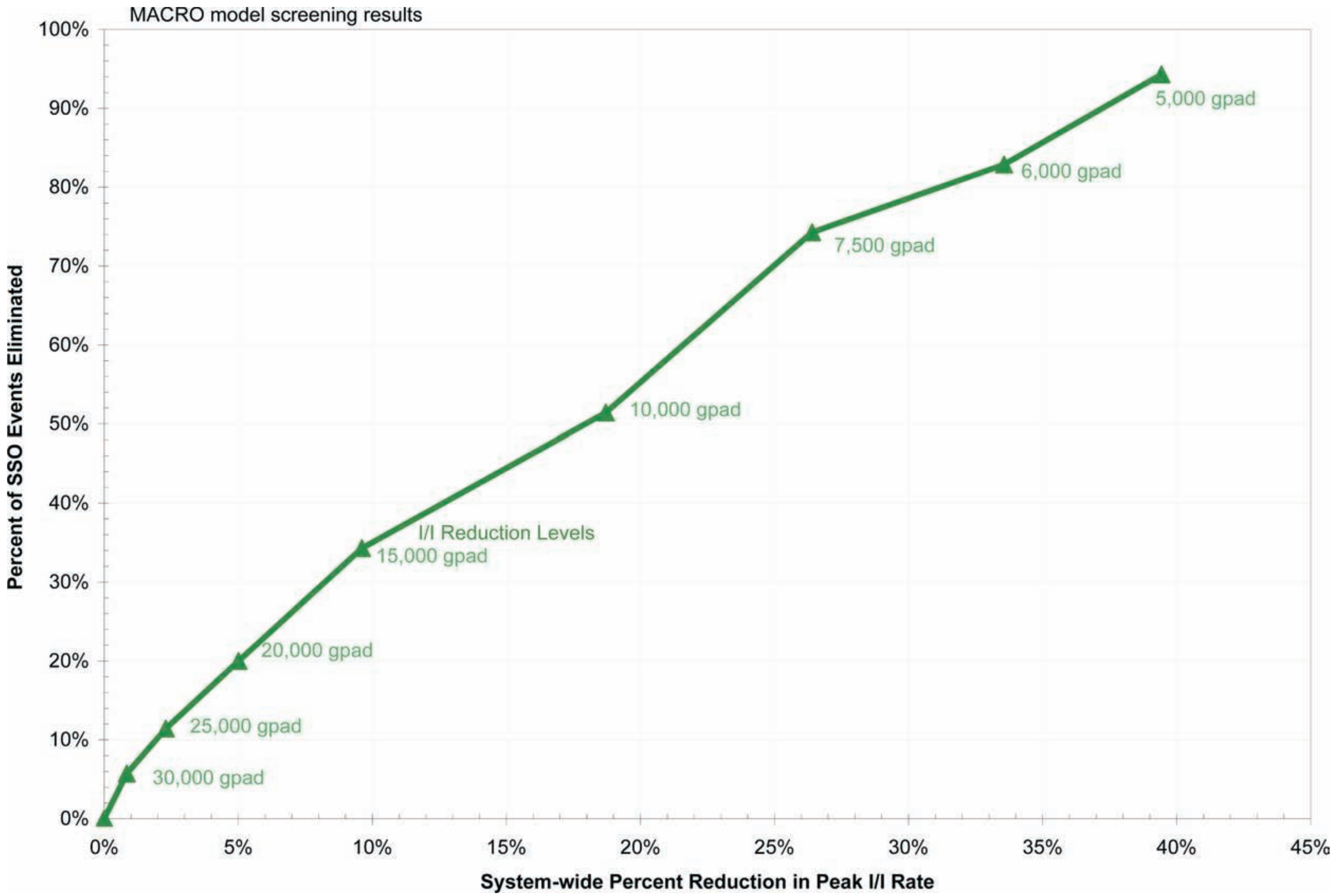

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FIGURE 3A-81

SSO EVENT PRODUCTION FUNCTION: I/I REDUCTION

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3A.4.7 Cost Benefit Curve

The initial slope of the cost benefit curve for SSO volume removed shown in Figure 3A-82 is relatively flat for the first level of I/I reduction (30,000 gpad). For progressively larger amounts of I/I reduction (with lower I/I target levels), the cost benefit curve becomes increasingly steep.

3A.5 Maximum Practical Sewer Separation for the Combined Sewer Service Area

3A.5.1 Introduction

The separation of combined sewers reduces or eliminates the volume of combined sewage (sanitary sewage mixed with stormwater) that is directly discharged into surface waters during a wet weather event. Sewer separation results in independent storm and sanitary sewer systems, thereby eliminating combined sewers and the potential for combined sewer overflows. However, sewer separation also results in a substantial reduction in the amount of polluted urban stormwater that is collected during most wet weather events in the combined sewers system and delivered to one of the treatment plants. Therefore, although the risk of combined sewer overflows is reduced by sewer separation, surface water quality is better protected with the use of combined sewers to collect urban stormwater. Nevertheless, sewer separation was evaluated as an alternative. Sewer separation can be implemented to varying degrees, from complete separation to various levels of partial separation. This section analyzes the implementation issues associated with complete sewer separation, including separation on private property. It also discusses other partial sewer separation technologies.

Although sewer separation is intended to eliminate combined sewer overflows, it will also affect all the other water quality indicators. Implementing this technology will reduce the volume of sanitary sewage and related pollutants entering surface waters, but at the same time will direct increased volumes of urban nonpoint stormwater flow and its related pollutants into surface waters unless the “first flush” from the newly separated areas is directed to the ISS or stormwater best management practices (BMPs) are installed to treat the stormwater runoff.

Currently, the combined sewers, which are owned by the city of Milwaukee and the village of Shorewood, are connected to the MIS system and to the ISS, both owned by the MMSD. At the inception of a wet weather event, the stormwater mixes with sanitary sewage and is directed into the MIS and the ISS, thus capturing the highly polluted first flush of combined sewage. As the event continues and combined sewage flows increase, MMSD may need to close tunnel gates to direct combined sewage to the surface waters. The decision to allow a combined sewer overflow is determined by evaluating the predicted size and duration of the event and permit requirements to reserve volume in the ISS to prevent separated sewer overflows.

Separation of the combined sewer systems will dramatically reduce the flow volume to the MIS and the ISS by diverting or allowing the majority of the stormwater from the CSSA to flow directly to the waterways.

3A.5.2 Prior Analysis of Complete Sewer Separation

The MMSD first evaluated the separation of combined sewers in the *Combined Sewer Overflow Facility Plan Element* (CSO FP) of the Milwaukee Water Pollution Abatement Program in a report dated June 1, 1980.(18) That plan evaluated a variety of sewer separation methods and implementation levels and developed detailed costs for sewer separation within the CSSA.

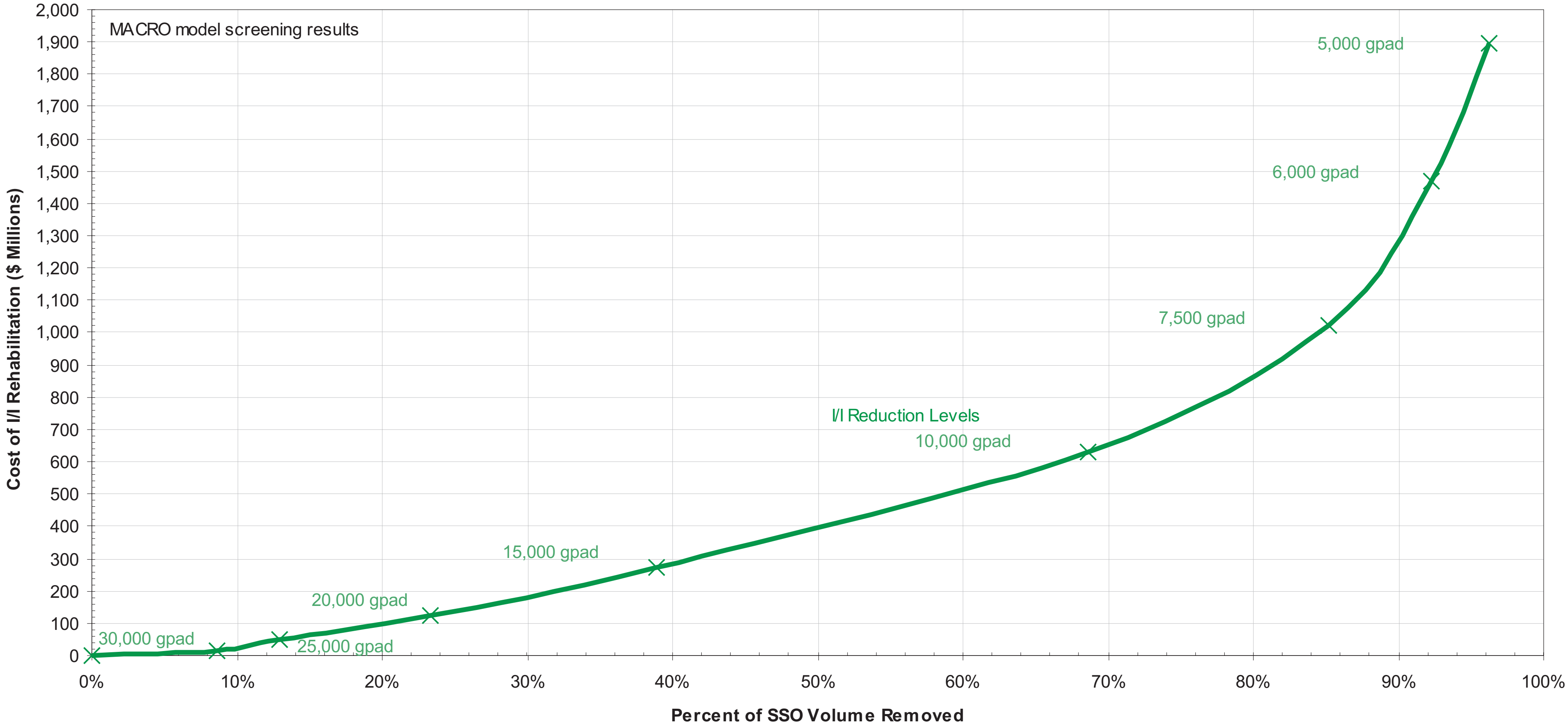

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FIGURE 3A-82
SSO VOLUME COST BENEFIT CURVE: I/I REDUCTION
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The CSO FP compared eight different separation schemes to determine the method of choice for application throughout the CSSA. The analysis evaluated both complete separation and partial separation for each of four different mainline sewer options:

1) Construct a new sanitary sewer and leave the existing combined sewer to function as a storm sewer.
2) Construct a new low pressure lateral and sanitary sewer and leave the existing combined sewer to function as a storm sewer.
3) Construct a new storm sewer and leave the existing combined sewer to function as a separate sanitary sewer.
4) Construct new sanitary and storm sewers and abandon the existing combined sewer system.

Complete separation would remove all stormwater from the sanitary system by changing the following for each building: separating the plumbing system, installing a new sanitary lateral, and constructing a new main line sewer to carry the separated flow. The existing lateral would be left to carry the storm flow.

Partial separation would remove street drainage from the sanitary system, but would exclude building separation, thereby leaving most roof and foundation drainage in the sanitary system. The existing lateral would remain connected to the sanitary system (except for the low pressure sewer option). Because some stormwater would remain in the sanitary system, these options could possibly need to consider overflow points to surface waterways, an option that would require WDNR approval in the discharge permit process. (Note that if these were to be designated as sanitary sewers, Wisconsin Department of Natural Resources would probably no longer allow the construction of new overflows.)

The CSO FP also evaluated several options for separating plumbing systems in residences and small commercial buildings:

- Interior methods included reconnecting downspouts to a new drain line suspended from the basement walls or ceiling or constructing a new building drain under the basement floor. The underfloor option was eliminated because it was too costly and could not guarantee that storm and sanitary flows could be completely separated.
- Exterior methods included reconnecting downspouts to a shallow storm sewer around the perimeter of the building, redirecting downspouts to discharge to grade, or rerouting downspouts to drain to the front of the building where they would be connected to the existing or new lateral. The latter two options were dropped as being impractical on a widespread basis.

Larger buildings were considered case by case.

Based on cost and water quality benefits (related only to the reduction in SSOs with no consideration given to stormwater quality impacts), the preferred sewer separation scheme was assumed to be complete separation with new sanitary sewers and included the following:

- Completely separate 100% of the CSSA by constructing about 2.3 million feet of sanitary sewer from 8 to 54 inches in diameter and 15 sanitary lift stations. Because the new sanitary sewers would be lower than the existing combined sewers, the lift stations would be required to allow lateral connections from both sides of the street and to avoid conflicts at crossings.

- Separate all building plumbing systems in the CSSA, assuming 80% would use the internal method (basement ceiling or wall system) and 20% the exterior method (shallow storm collector). All 60,600 residential and 2,750 commercial and industrial buildings within the CSSA would receive new sanitary laterals. Building separation also included adding sump pumps to the 10 to 20% of residences in the CSSA anticipated to have foundation drains.

3A.5.3 Definition of Sewer Separation for the 2020 Facilities Plan

In collaboration with representatives from the city of Milwaukee, the village of Shorewood, and the Wisconsin Underground Contractors Association, the 2020 technical team began the sewer separation analysis by reviewing the assumptions, design criteria, and cost criteria of the 1980 Combined Sewer Overflow Facility Plan (CSO FP).

The group achieved a consensus on the sewer separation technology to be evaluated in the 2020 FP by resolving the following issues:

- The complete sewer separation scheme preferred by the CSO FP was confirmed to include new sanitary sewers and new sanitary laterals to every structure. The existing combined sewer would remain and function as the storm sewer.
- Complete separation would require work on private property, with the installation of new sanitary laterals with a new connection to the building sewers and potential modifications to remove downspouts from the building sanitary sewers.
- Due to the narrow lots in the CSSA and the extensive street excavation required for the new sewers and laterals, it was assumed that the entire street would be replaced from curb to curb, including new sidewalks.
- The “first flush” from the storm sewers (existing combined) would continue to be captured by the existing network of MIS intercepting structures and diversion structures to the ISS. Modifications to the operation of structures would be required to implement a “first flush” operational mode.
- The central business district (CBD) would be excluded from the separation alternative on the premise that the work would be too disruptive to downtown streets and businesses.

Areas to be considered for separation were determined based on sewershed boundaries and land use data provided by SEWRPC and the city of Milwaukee. In general, sewersheds with less than 10% residential land use (defined as three stories or less) were excluded from consideration. Some non-residential sewersheds in the industrial areas south of downtown and in the Menomonee Valley, where separation could be accomplished relatively easily, were added to the separation analysis. Conversely, some residential areas just north of downtown were excluded to avoid the situation of a newly separated storm sewer flowing into a sewershed that would still have combined sewers.

The net result of this analysis shows that the area that would remain combined extends generally from McKinley Avenue on the north to National Avenue on the south, and from approximately 15th Street to Lake Michigan. Separation was considered in 105 of the 161 CSSA sewersheds, covering approximately 13,900 acres (21.7 square miles), or 89% of the total 15,610 acres (24.4-square miles) in the CSSA.

The files from CSO FP included detailed tabulations of building counts and required sanitary sewer lengths by size for each of 176 subareas within the CSSA. These were used to determine the approximate quantities required to separate the 127 subareas that corresponded to the 105 sewersheds now identified for separation.

However, the sewer design criteria developed for use in the 2020 FP are significantly higher than those used for the CSO FP, particularly the infiltration and inflow I/I allowance. Accordingly, while the total footage of sanitary sewer may be similar, there was some concern that the pipe size distribution could be significantly different when using the new criteria.

Five sample areas were selected for a detailed evaluation to develop a tabulation of sanitary sewer quantities using current design criteria to compare against the quantities from the CSO FP. The results showed that the total length of sewer required was within 3% of the quantities shown in the CSO FP tabulations, and, more importantly, the quantities for pipe sizes 12 inches and over were actually lower using the new criteria than the totals shown in the CSO FP data. The evaluation demonstrated that the pipe sizes shown in the CSO FP study were sufficiently conservative to allow their use as the basis for an updated sewer separation cost estimate.[f]

The sewer separation program would involve construction of 2.1 million feet (401 miles) of new sanitary sewer ranging from 8 to 54 inches in diameter, over 85% in the 8-inch size. Of the 15 lift stations identified in the CSO FP, 11 would still be required in the area identified for separation. All would be located adjacent to existing intercepting structures on the MIS. Accordingly, they would be merely lifting the flow to discharge to the MIS, and no significant length of force main would be required. Building separation would be required in 57,700 residential and 1,430 commercial structures.

3A.5.4 Production Function

The development of production functions are based upon separation of 89% of the combined sewer service area, thereby removing 89% of the CSOs in terms of annual volume. Sewer separation will result in a discrete reduction of flow to the ISS and a similar increase of the stormwater watershed component to the waterways for specified wet weather event inputs.

While the potential exists for implementing less than the 89% sewer separation concept presented here, the concept was to present the use of separation to the “maximum extent practicable.” However, since the connections to the ISS currently exist, it is assumed for water quality purposes that the first flush generated during a wet weather event will be captured by the ISS through operation of the existing diversion gates to the ISS.

Separation Costs

Combined Sewer Overflow Facility Plan Costs

The CSO FP estimated the construction cost of complete sewer separation in the entire CSSA at $382 million in 1978 dollars (ENR-CCI = 2700).(19) The detailed tabulations of separation quantities showed that the subareas where separation work would be required added up to 14,555 acres, representing an average construction cost of about $26,250 per acre. Escalating the costs

[f] Data and evaluation in 2020 FP project files.

to an ENR-CCI of 10,000 (projected to be in 2007) that is used for the 2020 FP gives a current equivalent cost of about $97,000 per acre.

During the review of the CSO FP, a separate analysis of separation construction costs per acre was conducted to determine whether there were patterns related to land use type or location within the CSSA. The analysis resulted in calculated unit costs for 79 areas of varying size throughout the CSSA, which varied from $7,840 to $46,710 per acre, with an average value of $25,740 per acre (all costs in 1978 dollars). No discernible land use patterns were evident in the unit costs, and the extreme values tended to be in small areas, where a small change in acreage could produce a large change in unit cost. Adjusting the average value to reflect 2007 dollars yields an equivalent current construction unit cost of $95,333 per acre, which is very close to the overall average for the entire CSSA discussed above.

More specifically, the detailed quantity and cost tabulations in the CSO FP showed that the estimated total construction cost for separation in the 105 subareas included in the 2020 FP separation program totals $348.4 million (1978 dollars). Over the total area of 13,900 acres, this yields a 1978 construction unit cost of $25,000 per acre, nearly equivalent to the values discussed previously.

Applying these unit costs to the 13,900 acres now considered for separation yields a total projected construction cost ranging from $1,325,000,000 to $1,348,300,000 at ENR = 10,000.

Unit Construction Cost Methodology

Tables of current construction costs for installation of sewers were prepared and used to estimate the construction cost of the separation program. Unit costs per foot of installed sanitary sewer included:

- Pipe and installation
- Excavation, hauling and trucking
- Backfill
- Removal of pavement and replacement in kind
- Removal of curb & gutter, and replacement in kind
- Trench support
- Dewatering
- Lawn and sidewalk replacement
- Manholes

These unit costs, when applied to the estimated quantities of sewers ranging from 8 to 54 inch diameter, generated a value of $902.2 million, including an allowance of 15% for handling typical residential area utility conflicts during construction. Adding private property costs, estimated at $727.2 million, and lift stations, at $16.9 million, yields a total construction cost of $1,646.3 million. Over the 13,900 acres in the area considered for separation, the total construction cost works out to $118,400 per acre. (ENR-CCI = 10,000)

Comparison of Methodologies and Total Project Cost

The unit cost method results in a total separation construction cost that is 26% higher than the scaled-up values from the CSO Facility Plan. The unit cost table method is considered more reliable, since it is based on current cost data and economic factors.

The total project cost for sewer separation was evaluated using a contingency of 25% (as used for the other technologies in this report) and 50% of construction cost to accommodate the uncertainties inherent in the widespread implementation of sewer separation throughout the CSSA. Also included in total project cost are engineering design and services during construction, estimated at 30% of base construction cost. The resulting total project cost with a 25% contingency is $2.8 billion or $200,000 per acre and with a 50% contingency is $3.034 billion, or $220,000 per acre. (ENR-CCI = 10,000)

The cost summary is shown in Table 3A-16.

Comparison with Sewer Separation Projects in Other Municipalities

Major sewer separation projects have been initiated in recent years in Atlanta, Georgia and Lansing, Michigan.(20,21)

Atlanta's program was originally estimated at $989 million construction cost (ENR-CCI = 9360) for 12,200 acres, or $82,417 per acre ($88,052 per acre at ENR = 10,000). However, initial contracts that were bid came in at 78% over bid estimate. If that trend continues, the cost per acre increases to $146,671 ($156,700 per acre at ENR-CCI = 10,000) and $203,000 per acre including 30% costs for engineering, administration, etc. Considering that this value does not include work on private property (which represents about 44% of the estimated construction cost in Milwaukee), it compares favorably with the $210,000 to $250,000 per acre estimated above.

Lansing's 30 year program of 203 miles of new sanitary sewers was estimated at $353 million (ENR-CCI = 10,000). However, comparison with Lansing costs is less valid for a variety of reasons.

- The Lansing costs do not include utility relocations, since those costs are assumed by the utilities
- Work on private property is limited
- All labor costs in Lansing are lower by about 25%, not only for open cut sewer construction but for all construction types

All costs at Milwaukee ENR CCI (June 2007) = 10000 (projected)

Sanitary Sewer:

PIPE SIZE (Inches)	**TOTAL LENGTH** (Feet)	**UNIT PRICE ($/Lin Ft)** "Residential"	**TOTAL COST** "Residential"	**$/LF** "Mixed"	**TOTAL COST** "Mixed"
8	1,805,900	$413.00	$745,836,700	$431.00	$778,342,900
12	147,800	$436.00	$64,440,800	$455.00	$67,249,000
15	62,630	$440.00	$27,557,200	$460.00	$28,809,800
18	35,250	$553.00	$19,493,250	$577.00	$20,339,250
21	19,720	$570.00	$11,240,400	$595.00	$11,733,400
24	12,920	$581.00	$7,506,520	$606.00	$7,829,520
27	10,380	$723.00	$7,504,740	$755.00	$7,836,900
30	5,240	$752.00	$3,940,480	$785.00	$4,113,400
36	11,080	$819.00	$9,074,520	$854.00	$9,462,320
42	4,160	$884.00	$3,677,440	$923.00	$3,839,680
48	450	$956.00	$430,200	$997.00	$448,650
54	1,440	$1,094.00	$1,575,360	$1,141.00	$1,643,040
	2,116,970 LF				
	400.9 mi				
Construction Cost - New Sanitary Sewer:			**$902,278,000**		**$941,648,000**

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TABLE 3A-16 SHEET 1 OF 2

SUMMARY OF CONSTRUCTION COSTS FOR SEWER SEPARATION

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	TOTAL AMOUNT		UNIT PRICE	TOTAL COST "Residential"	TOTAL COST "Mixed"
Laterals:					
Wye connections	60,554	ea	$250	$15,139,000	$15,139,000
Laterals within ROW	60,554	ea	$4,000	$242,216,000	$242,216,000
Res Lats (Front)	54,818	ea	$2,000	$109,636,000	$109,636,000
Res Lats (Back)	2,878	ea	$9,000	$25,902,000	$25,902,000
Res Bldg Separation	57,696	ea	$5,000	$288,480,000	$288,480,000
Risers	8,560	ea	$560	$4,794,000	$4,794,000
Commercial Bldg Sep	1,429	(LS)		$41,000,000	$41,000,000
Construction Cost - New Laterals:				**$727,167,000**	**$727,167,000**
Construction Cost - Lift Stations:				**$16,873,700**	**$16,873,700**
TOTAL CONSTRUCTION COST - SEWER SEPARATION				**$1,646,319,000**	**$1,685,689,000**
CONTINGENCY			**50%**	**$823,159,500**	**$842,844,500**
ENGINEERING & SERVICES DURING CONSTRUCTION			**30%**	**$493,895,700**	**$505,706,700**
TOTAL ESTIMATED SEWER SEPARATION COST:				**$2,963,374,200**	**$3,034,240,200**
TOTAL ESTIMATED SEWER SEPARATION COST only 25% CONTINGENCY:				**$2,675,268,375**	**$2,739,244,625**

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TABLE 3A-16 SHEET 2 OF 2

SUMMARY OF CONSTRUCTION COSTS FOR SEWER SEPARATION

3A.5.5 Cost Benefit Curve

In Figure 3A-83, the sewer separation cost is shown as a dashed line in that only the 89% level of analysis was completed. The dashed line shows the effect of less than 89% area separation assuming a linear function. This figure also contains the other technologies that have been evaluated for CSO reduction, as well as the cost benefit data for the other sewer separation technologies (partial sewer separation – downspout disconnection and partial sewer separation – downspout disconnection with catch basin flow). Figure 3A-84 zooms in on the first 20% of CSO reduction.

3A.5.6 Summary

Separation of sewers is a proven technology that will effectively isolate the sanitary derived wastewater component from the stormwater generated component. Complete separation on private property provides a new sanitary lateral to each structure, thereby minimizing the lateral as a source of infiltration.

While sewer separation will effectively eliminate the stormwater component of combined sewer flow from capture by the ISS and treatment, the existing connection of the combined sewer system to the ISS does afford the capture of the first flush by the ISS with the operation of the existing system. Thus, with the exception of that first flush capture, sewer separation will save the costs of pumping and treatment. The pollutants associated with the remaining stormwater component will no longer be treated, but will be discharged into the area waterways. The analysis of sewer separation as a screening alternative includes the capture of the first flush. See Appendix 9A, *Screening Alternatives* of the *Facilities Plan Report* for more information.

3A.6 Sewer Separation – Shorewood Project

3A.6.1 Introduction

Reducing peak wet weather flows entering the combined sewer system will reduce the frequency and volume of CSOs, and may allow the ISS to accept additional volume from the SSA.

To effect such reductions, the village of Shorewood, in partnership with MMSD, instituted a Wet Weather Flow Volume and Peak Management Project in early 2003, seeking to provide protection from basement backups without requiring conveyance capacity enhancements that would increase the village's peak rate and volume discharges to MMSD facilities. This Plan included an element dealing with runoff management, which considered means of reducing runoff volume and peak discharge rates from both rooftop and surface sources:

- In the case of rooftop runoff, the principal method of volume reduction considered was the disconnection of downspouts in the partially separated areas of the village north of East Capitol Drive, where storm sewers are already available.
- Peak roof runoff rate reduction was evaluated in remaining areas of the CSSA, principally through the consideration of downspout disconnection in conjunction with rain barrels and rain gardens.

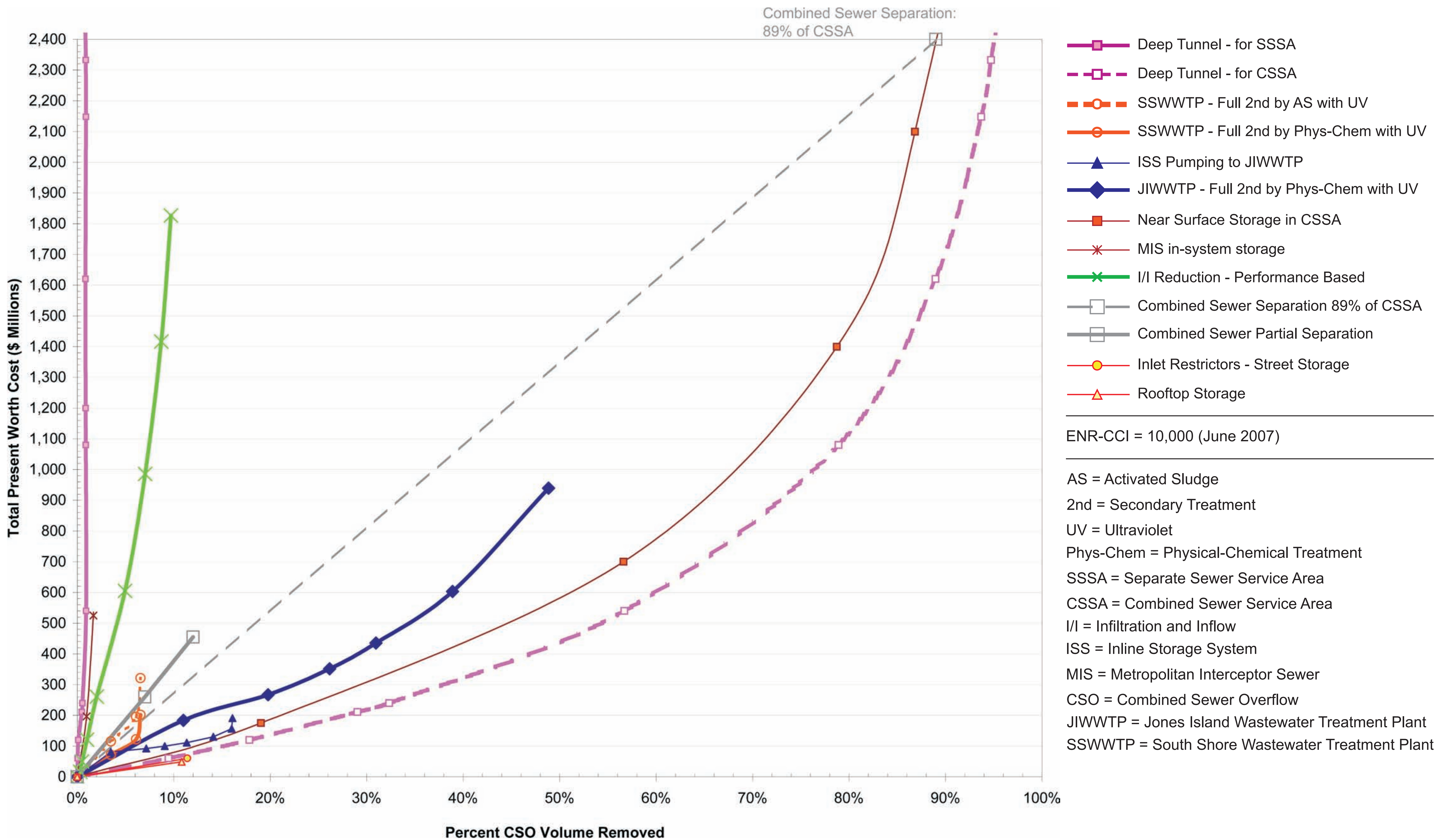

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FIGURE 3A-83
CSO VOLUME COST BENEFIT CURVES: SEWER SEPARATION TECHNOLOGIES AND OTHER TECHNOLOGIES

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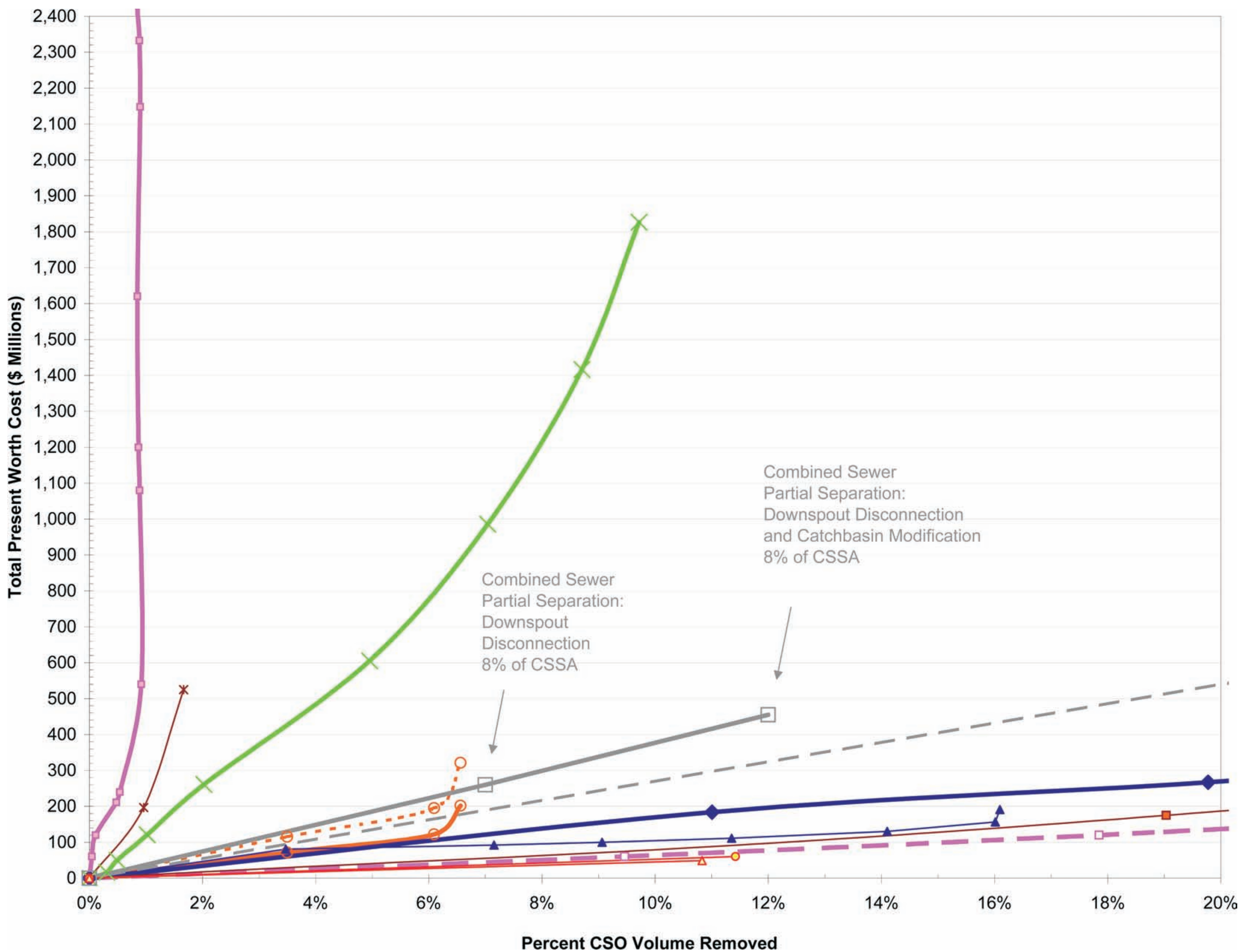

ENR-CCI = 10,000 (June 2007)

AS = Activated Sludge
2nd = Secondary Treatment
UV = Ultraviolet
Phys-Chem = Physical-Chemical Treatment
SSSA = Separate Sewer Service Area
CSSA = Combined Sewer Service Area
I/I = Infiltration and Inflow
ISS = Inline Storage System
MIS = Metropolitan Interceptor Sewer
CSO = Combined Sewer Overflow
JIWWTP = Jones Island Wastewater Treatment Plant
SSWWTP = South Shore Wastewater Treatment Plant

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FIGURE 3A-84
CSO VOLUME COST BENEFIT CURVES: SEWER SEPARATION TECHNOLOGIES AND OTHER TECHNOLOGIES (ZOOM IN ON FIRST 20% OF CSO REMOVAL)

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- Reduction in the volume of surface runoff reaching the combined sewers was evaluated by considering the connection of additional catch basins to the storm sewer system. This method was limited to areas within or adjacent to the partially separated portion of the CSSA where storm sewers are already available.
- Reduction in peak rate surface runoff was evaluated by considering the installation of catch basin flow regulators to delay peak flows, temporarily storing runoff in the streets.

A report of the study design was prepared by a Shorewood consultant in 2003.(22,23)

In June 2004, MMSD authorized an agreement with Shorewood to implement the project, with MMSD's cost share not to exceed $307,500. The same resolution also awarded up to $60,000 to the University of Wisconsin – Milwaukee for assistance with the development of a plan to make the UWM campus a zero discharge zone.(24) The plan involved a number of stormwater BMPs, including downspout disconnection, green roofs, rain gardens, and porous pavement. The Stormwater Master Plan was scheduled for completion in January 2006.

3A.6.2 Design Factors

For downspout disconnection, design factors that were considered related to the location of the downspouts on the house and the ability to separate their flow from the sanitary connections. In some cases, a new perimeter collection system would be needed to direct the downspout connections to the front of the house, or the gutters would need to be resloped to flow toward the front of the house. The availability of capacity within the existing storm sewer was considered.

There were no design factors related to rain barrels other than identifying those downspouts where their installation was feasible. Rain gardens would be sized based on the amount of tributary roof drainage area, and their location is subject to topographic and space restrictions on the subject property.

The extension of existing storm sewers was subject to the usual design factors considered in the design of drainage facilities. However, in this Shorewood Project, the availability of additional capacity in the existing trunk sewers and outfalls was also considered. This made the application of this technology site-specific.

Implementing the catch basin regulator system required intersection modifications to raise the street grade at the downstream end of the block to create the temporary storage pool. The system's applicability therefore would therefore be limited to areas where street slopes are flat enough that the raised intersection would allow sufficient volume to pool in the street without overtopping the curb or generating traffic hazards. Because of the inconvenience imposed on motorists by temporary flooding, the system would only be implemented on residential streets with low traffic volumes. These again are site-specific considerations.

The objective of the Shorewood Project was to remove the maximum amount of stormwater from the combined sewer system. The study design report included the following observations:

1) Downspout disconnections of about 11 acres of roof area in the CSSA would reduce runoff by 312,000 gallons per inch of rainfall
2) Storm sewer construction in about 15 acres of the CSSA would reduce runoff by 165,000 gallons per inch of rainfall

3) An intensive wet weather flow management plan including BMPs and downspout disconnections would reduce runoff by 539,000 gallons per inch of rainfall

3A.6.3 Experience

With the exception of the catch basin regulator system, all of the practices described above are common and effective means of controlling and disposing of stormwater and are within the collective local experience to construct and maintain. The technology of catch basin regulators and temporary street flooding is untested in the Milwaukee area, but has been successfully employed in other parts of the country, including Chicago, Skokie, and Wilmette, Illinois.

3A.6.4 Production Function

Information regarding technologies that control roof runoff, including downspout disconnection, rain barrels, and rain gardens, is discussed in Chapter 4, *Nonpoint Source Technology Analysis* of this report.

The practice of rerouting catch basins to existing or extended storm sewers is site-specific in the volume reduction obtained, and therefore does not lend itself to production function theory.

The technology of catch basin regulators and temporary street storage of runoff also does not lend itself to the development of a production function. The complex hydraulics and variables related to drainage areas, topography, runoff rates, overland flow, and bypass flows are unique to the particular drainage area and make it difficult or impossible to predict the peak flow rate reduction to be obtained. The effectiveness can only be evaluated on a total project basis, assuming full implementation wherever it is feasible within a particular combined sewershed.

3A.6.5 Cost Function and Cost Benefit

Project costs for Shorewood's catch basin regulator and street storage program are not yet available, nor have peak flow rate reductions been evaluated. Therefore, cost functions cannot be developed. The cost benefit evaluation of roof runoff control measures is discussed in Chapter 4 of this report. The cost benefit of extending existing storm sewers must be evaluated on a case by case basis, as both costs and benefits are unique to each application. For the catch basin regulator system, the cost benefit analysis consists of only a single point, representing the overall project in Shorewood.

3A.6.6 Summary

All of the practices discussed here have value in reducing conveyance and treatment costs, optimizing the collection system capacity, reducing CSO volume and frequency, and, in conjunction with storm sewer BMPs, reducing the pollutant loadings to the waterways. However, the two unique technologies presented have limited application within the CSSA beyond the area where they have been implemented in Shorewood.

Rerouting of catch basins to existing or extended storm sewers is applicable only within or adjacent to partially separated areas where storm sewers already exist. Such opportunities are very limited within the Milwaukee portion of the CSSA.

For the temporary street storage technology, the requirement for flat slopes and the need for intersection modifications, along with the negative effects of temporary street flooding make it unlikely this technology would find widespread application within the city of Milwaukee. In addition, during intense or extended storms, runoff in excess of the stored volume would flow out of the pool, increasing the load on downstream catch basins and creating the potential for increased flooding on private property or for creating street flooding problems where none now exist. Depending on the duration and timing of the peak flow, this could also negate the peak flow reduction the system was intended to provide. Nevertheless, any opportunity that can be identified as a potentially significant source of stormwater peak flow reduction should be pursued on a site by site basis to determine if it is cost effective to implement.

3A.7 Partial Sewer Separation / Downspout Disconnection

3A.7.1 Introduction

Reducing peak wet weather flows entering the combined sewer system will reduce the frequency and volume of CSOs and may allow the ISS to accept additional volume from the SSA. Disconnecting residential downspouts within the CSSA will help accomplish this by redirecting the roof runoff to a new dedicated storm or express sewer that then discharges to a nearby waterway. This discussion considers only the reduction in peak wet weather flow rates and annual volume and does not take into account water quality issues related to direct discharge of stormwater to local waterways.

Historically, all residential and commercial buildings in the Milwaukee CSSA were required under Chapter 225 of the city of Milwaukee Code of Ordinances to connect their downspouts to the combined sewer. Residential garages and other accessory buildings of less than 1,000 square feet were not required to have gutters and downspouts (under Chapter 252-71 of the Code of Ordinances), and most of them probably do not. Milwaukee revised Chapter 225 in December 2001 to allow surface discharge of roof runoff from all buildings within the CSSA and from single- and two-family residences throughout the city. This revision was made primarily to accommodate the pilot programs sponsored in recent years by MMSD. However, to avoid creating drainage or safety problems on the subject property or adjacent properties, the ordinance contains strict requirements for the discharge location and direction that are often difficult to meet on the narrow lots that predominate in the CSSA. Furthermore, redirecting the downspout to the lawn is less that 100% effective in reducing the runoff volume entering the combined sewers, because much of this runoff volume still enters the system through the street catch basins. Therefore, to obtain meaningful volume reductions from downspout disconnection, a stormwater conveyance system is an essential part of this technology.

3A.7.2 Design Factors

While the actual disconnection of downspouts is implemented at the individual private property level, the need for a public conveyance system requires that the production and cost functions evaluate implementation throughout a “typical” residential area. The evaluation considered a 16-block area, four blocks on a side.

Two alternative configurations were developed for the small diameter roof drainage collection system within the 16-block area: one with connections from the front of each house, the second from the back (alley side).

For the front-side option, the collector sewer would be within the street right-of-way but outside the curb to avoid interfering with the sewers and other utilities in the street. Many streets in the CSSA are already crowded with utilities, often including multiple combined sewers and CSO sewers, some of which are very large. Other advantages to placing the roof drainage collectors behind the curb rather than in the pavement include: less disruption during construction, reduced restoration costs, and shorter lateral connections from the houses. Disadvantages of this arrangement include: collectors must still cross water and sewer laterals at each property, conflicts with gas mains and electric or communications lines are more likely, and the collectors will only serve one side of the street, requiring twice the total length of collector sewer.

Advantages of the alley location include: underground utility conflicts are likely to be limited to the ends of each block and the collector sewer can serve houses on both sides of the alley. The alley location also provides an opportunity to increase the amount of roof area connected to the system by adding gutters and downspouts to the garages and connecting them to the new storm lateral. Disadvantages of the alley location include: longer laterals from each house and tighter clearances and overhead electric lines, which will make construction more difficult. This distinction applies only if garages are *not* included. If they are, lateral length turns to an advantage for the alley location because the lateral is actually shorter going to the alley since the front setback is excluded. Because houses are generally set near the front of the lot, access to residents' garages will be restricted during construction. Redirecting the roof drainage to the back of the lot will also likely require more extensive modifications to the existing gutters and underground roof collectors at many houses.

Regardless of the collector sewer configuration, it was assumed that the roof runoff collector sewers would connect to a conveyance sewer located in the street at the low end of each block, which would increase in size as the cumulative tributary area increased toward the downstream end of the conveyance system.

A comparison of the two collector sewer configurations showed that the total length of collector was the overriding cost factor, making the alley system the less expensive of the two. For the private property portion of the work, each house would have to be evaluated to determine the amount of work required to redirect the roof drainage to the new collector system. Factors to be considered include the configuration and locations of the gutters and downspouts and their connection to the existing combined lateral, and the ability to separate their flow from the sanitary connections. In some cases, a new underground perimeter collection system may be needed to direct the downspout connections to the back of the house, or the gutters may need to be resloped to flow toward the back of the house.

The collection and conveyance system would be designed similar to any other drainage system, with the layout and pipe sizes determined by the topography of the area and the total tributary drainage area.

A field assessment aided by review of available maps of the CSSA was conducted to identify areas where implementation of this system would be most practical. The survey looked for residential areas where a street or alley either borders directly on or drains to a waterway, which would allow the new large trunk storm sewers to run directly into a waterway without crossing wide expanses of park land or major street intersections. Because many areas adjacent to the waterways tend to be "atypical" (either heavily industrial or parkways), the survey determined that, as a practical matter, implementation of this technology is probably limited to no more than 8% of the CSSA.

3A.7.3 Experience

Downspout disconnection is a common, effective and inexpensive method of removing wet weather flow from the combined sewer system. It involves common and proven construction methods that will effectively redirect the roof runoff to the waterways. However, the installation of a new lateral or perimeter collection system in a five foot deep excavation adjacent to the house will be difficult on narrow lots.

3A.7.4 Production Function

The volume of runoff removed from the combined sewer system by disconnecting downspouts varies with the roof area as a percentage of gross area, reflecting the development density in a given area. Examination of aerial photographs of residential areas within the CSSA reveals that total roof area including garages ranges from 15 to 26% of the total area. Therefore, the maximum estimation of CSO removal is:

8% (of the CSSA area) x 26% (area removed) = 2.0% CSO Reduction

Excluding garage roofs from the downspout disconnection program would reduce the total roof area and resulting volume removals by approximately 20 to 30%.

The most economical areas in which to implement downspout disconnection would be closest to the receiving waterways. As the area of implementation expands away from the waterways, their tributary areas increase and the conveyance sewers increase in size. This increases the cost of the conveyance system and greatly complicates its construction, as the streets closer to the waterways already tend to have the larger combined sewers.

3A.7.5 Cost Function

Capital costs are estimated at about $129,000 per acre (ENR-CCI = 10,000). This estimate is based on the following assumptions:

- Each block has an average of 36 structures, for a total of 576 residences over the entire 16 block area.
- A 12-inch roof runoff collector sewer in the alley would serve houses on both sides of the alley.
- The roof gutters and downspouts would be reconfigured on every structure, including garages, to redirect the roof drainage to the new lateral.
- A new 6-inch PVC lateral would be installed on every property to convey the roof runoff from the house and garage to the new collector sewer. The average lateral length would be 80 feet from the back of the house to the alley, installed at a nominal depth of 5 feet. Some properties would also require a new underground perimeter collection system around the house where the gutters cannot be directed to a collection point near the lateral.
- Collector sewers would connect to a 15-inch storm sewer at the low end of each block, extending to the north-south street at the edge of the area. The 18-inch conveyance sewer would then extend three blocks north or south to the corner of the area. It is assumed that another block of 18-inch conveyance sewer would extend to the discharge point at the waterway. Actual sizes of the sewers would depend on the topography and size of the tributary roof drainage area.

The cost summary is shown in Table 3A-17.

Based on the premise that the 8% of the CSSA where this system could be implemented represents residential areas adjacent to waterways, this cost is assumed to include the conveyance system all the way to its discharge point at the waterway. For areas more than about ½ mile from the waterway, the cost of the additional conveyance system would have to be added. The length and size of sewers required would have to be determined specifically for the area under consideration.

3A.7.6 Cost Benefit Curve

Figure 3A-84, shown previously, shows the cost benefit curve for this technology as a "box" on the graph.

3A.7.7 Summary

The appeal of downspout disconnections lies in the intuitive notion that they are easy to disconnect from the combined sewer system and that the stormwater conveyance system required to remove the runoff would be smaller than that required under complete separation. However, the conveyance system would still be large enough to result in high construction costs and significant construction difficulties if this technology were to be implemented throughout the CSSA. Limiting its application to the 8% of the CSSA where it would be most practical means the maximum impervious area that would no longer contribute runoff to the combined sewers would be 8% x 26% (the maximum observed roof coverage) = 2.0% of the gross area of the CSSA.

Figure 3A-84 shows that this technology is not cost competitive with other CSO reduction technologies such as tunnel storage or additional treatment capacity.

3A.8 Technology: Partial Sewer Separation / Downspout Disconnection and Catch Basin Modification

3A.8.1 Introduction

This technology is a variation of the "partial sewer separation / downspout disconnection" technology. It uses the potential value of upsizing the new storm sewer to accommodate excess flows from the street runoff. Such a feature would reduce the flows to the ISS, enabling the ISS to accept an equivalent amount of flow from other sources. This technology, as noted in the discussion on "partial sewer separation/downspout" technology, does not take into account the water quality issues related to the storm runoff component that reaches the waterway.

3A.8.2 Design Factors

This technology will direct some street runoff that otherwise would flow to the ISS to the nearest waterway through an enlarged storm sewer in the sidewalk area, but only after the first flush from the street runoff has been captured by the combined sewers.

ENR-CCI = 10,000

Item	Diameter (inches)	Downspouts disconnected w/ inlet connection: LF/unit	Cost: Unit Price	Cost: Total Cost	Downspouts disconnected: LF/unit	Cost: Unit Price	Cost: Total Cost
1 Manholes	48	36	*$3,500.00*	$126,000	20	*$2,000.00*	$40,000
2 Inlets [4]		36	*$2,500.00*	$90,000	0	*$2,500.00*	$0
3 Pipe from inlet to new sewer [1]	24	360	*$100.00*	$36,000	0	*$100.00*	$0
4 Roof lateral connections to the new stormsewer [5]		36	$10,000.00	$360,000	36	$10,000.00	$360,000
5 Street restoration [2, 6]		1,260	*$119.89*	$151,058	1,044	*$22.00*	$22,968
SUBTOTAL 1 (cost per city block excluding storm sewer)				$763,058			$422,968
(costs for 4x4 city block area)				**$12,208,932**			**$6,767,488**
6 Storm Sewer [7]	12	19,600	*$97.00*	$1,901,200	9,600	*$80.00*	$768,000
7 Storm Sewer [7]	15	2,530	*$100.00*	$253,000	4,600	*$100.00*	$460,000
8 Storm Sewer [7]	18	3,160	*$109.00*	$344,440	2,480	*$110.00*	$272,800
9 Storm Sewer [7]	21	1,260	*$118.00*	$148,680	0	*$118.00*	$0
SUBTOTAL 2 (storm sewer costs for 4x4 city block area)				**$2,647,320**			**$1,500,800**
SUBTOTAL 3 (SUBTOTAL 1 + SUBTOTAL 2)				**$14,856,252**			**$8,268,288**
CONTINGENCY 50% [3]				**$3,714,063**			**$2,067,072**
TOTAL (for 4x4 city block area)				**$18,570,315**			**$10,335,360**
TOTAL (per acre)				**$232,129**			**$129,192**
TOTAL (8% of CSSA)				**$292,965,289**			**$163,050,639**
Engineering and Administration @ 30%				**$87,889,587**			**$48,915,192**
Total project Cost				**$380,854,876**			**$211,965,831**

Notes:
1) Assume Concrete Pipe, distance from Inlet to new storm sewer is 40 ft. Costs from RS Means 2000.
2) This assumes worst case scenario that all roadway in the typical area will need to be restored.
3) Contingency includes utility relocation and sidewalk restoration.
4) 6 4-ft-deep inlets per city block. Price from Get a Quote.net.
5) Assume 40 lots per city block; includes lateral, and replacement of downspouts and gutters.
6) Restoration for 2 12-ft-wide lanes. Trench is 4 ft wide and 2 ft deep on a 1:1 slope. Refer to Assemblies Costs Tables in RS Means 2000.
7) Assume concrete pipe. Refer to RS Means for pipe excavation and backfill price. Refer to Get a Quote.net for pipe price itself.

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TABLE 3A-17
COST FUNCTION: 2020 BASELINE PARTIAL SEPARATION WITH AND WITHOUT CATCH BASIN INLET (DOWNSPOUTS ONLY AND DOWNSPOUTS WITH CATCH BASIN)

Because the essence of this technology is a direct connection between the new storm sewer and the street inlets, the new storm sewer would be located in the sidewalk area, rather than in the alley in the back of the residence. The new storm sewer would need to be increased in size to accommodate a certain portion of the street-generated runoff, but not so large that it would exceed the sidewalk area otherwise cost considerations would make the technology cost prohibitive. Therefore, the storm sewers would be limited to a size that would not exceed the bounds of the sidewalk. Additionally, this variation of partial sewer separation would only be applied to those areas in proximity to the watercourses to minimize the construction costs of the storm sewers. For that reason, the technology would be implemented in only 10% of the CSSA.

The modified inlet structure would be construed to accept varying amount of street/curb generated runoff, but the factor limiting the flow volume would be the requirement that the new storm sewer remains within the boundary of the sidewalk. For that reason, the amount of flow captured was assumed to be larger than the quantity generated by the rooftops, or 3% of the total CSSA area.

3A.8.3 Experience

The installation of the modified inlet structure in the curb and overflow pipe to the new storm sewer involves the use of common materials and traditional construction methods.

3A.8.4 Production Function

The development of variable production functions is applicable for this technology as it is defined as removing 3% of the CSSA. The downspout disconnection will result in a discrete reduction of flow to the ISS and a similar increase of stormwater to the waterways for specified wet weather event inputs. No potential exists for varying or otherwise implementing anything but the defined partial sewer separation concept.

3A.8.5 Cost Function

Similarly, development of variable cost functions is not applicable for this variation of downspout disconnection because this particular application of partial sewer separation was developed with only one cost value.

However, it is useful to understand the derivation of the costs for this partial sewer separation concept and the associated assumptions and qualifications. The cost derivation is similar to that prepared for the "downspout disconnection" alternative, with the following changes:

- The sewer work will be done in both the sidewalk and in street areas rather than in just in the sidewalk
- Costs are included for the installation of inlet structures and overflow pipes
- Cost of storm sewers are increased by 40% to account for the larger diameter storm sewer pipe diameters required.

Costs were estimated at $381 million (ENR-CCI = 10,000), which do not include the costs of large trunk sewers. The derivation of the costs for this partial sewer separation concept is contained in Table 3A-16 in the previous section.

3A.8.6 Cost Benefit Curve

Figure 3A-84, shown previously, shows the cost benefit curve for this technology as a “box” on the graph.

3A.8.7 Summary

This consideration of this technology evolved from the “partial sewer separation / downspout disconnection” technology. The basic idea for this technology is that, since a new storm sewer will be placed in front of every house, the sewer sizes should be increased to accept street runoff that would otherwise go to the ISS, thereby increasing its availability for flows from other sources. The application of this technology is limited to those areas close to the receiving waterway to minimize costly storm sewers that would be larger than considered for the “partial sewer separation / downspout disconnection” in order to accept the additional street runoff.

Figure 3A-84, shown previously, shows that this technology is not cost competitive with other CSO reduction technologies such as tunnel storage or additional treatment capacity.

3A.9 Partial Sewer Separation / Stormwater Disconnection (Rust/Harza Study)

3A.9.1 Introduction

Selective disconnection of stormwater sources from MMSD’s collection system within the CSSA will reduce the stormwater flows to the ISS. Application of appropriate BMPs to achieve treatment of the stormwater that subsequently flows to the waterways will reduce pollutant loadings as well.

A study conducted by Rust/Harza in 2002 identified 16 locations where storm sewers connect to MMSD’s conveyance system downstream of the MIS intercepting structures, resulting in direct contribution of stormwater to MMSD’s system under certain conditions. Some of these connections are between the intercepting structure and the MIS, while others connect to the near surface collector between the diversion structure and the ISS. Review of nine of the 16 locations indicated that the potential reduction in runoff volume was insufficient to warrant the capital cost of disconnection, leaving seven sites that offered the greatest potential for stormwater removal and low flow treatment.(25)

3A.9.2 Design Factors

Two distinct approaches were developed in the Rust/Harza study for each of the seven sites: 1) the wastewater treatment plant approach, where low flows from the “first flush” would be diverted to JIWWTP and the remaining flow discharges directly to the waterway, and 2) the BMP approach, where all stormwater flow would be discharged to the waterway after passing through either structural or non-structural BMPs to remove pollutants.

For the BMP approach, each site (and its tributary basin) was considered individually to determine the most appropriate type of BMP, based on cost and pollutant removal effectiveness. Selecting from a variety of structural and non-structural BMPs, a customized list of BMPs was developed for each location. Structural BMPs included catch basin inlet filters, in-line treatment systems, wet detention ponds, bioretention systems, porous pavement, green roofs, rain gardens, rain barrels, and green parking lots. Non-structural BMPs included pavement sweeping, catch basin cleaning, leaf collection, urban housekeeping, vegetation management, erosion control, and public education.

3A.9.3 Experience

The WWTP approach would involve the disconnection of stormwater sources from the conveyance system by constructing a low flow diversion structure. This is a simple, proven and effective technique in reducing runoff volumes to the MMSD system. This technique would limit flows to the MMSD system to the low flow stormwater component, while higher flows would be directed to the waterway.

Under the BMP approach, measures considered are commonly used structural and non-structural techniques that require regular maintenance to achieve the highest practical pollutant removal. All of these measures are currently in use to varying degrees in the Milwaukee area and are within the operation and maintenance capabilities of local governments and property owners.

3A.9.4 Production Function

The effectiveness of this technology in reducing stormwater volumes entering the MMSD system and in reducing sediment load on the receiving waterway was evaluated individually at each site. While the resulting points could be compiled into a curve, the defining parameters are unique to each site and drainage area, so this technology does not have a true production function.

Full implementation at all seven sites would remove about 31 million gallons annually from MMSD's system, or about 3% of the average annual CSO volume, and, in conjunction with the BMPs, would reduce the sediment load to the waterways by 14,000 pounds annually based upon the Rust/Harza report.

3A.9.5 Cost Function

Similarly, variable cost functions are not applicable for this technology because the implementation costs are unique to each site. The total capital cost for all seven sites exceeds $6.6 million (not including contingencies), with corresponding pumping and treatment cost savings of $22,500 annually according to the Rust/Harza report.

3A.9.6 Cost Benefit Curve

The Rust/Harza study prioritized the seven sites according to their cost effectiveness in removing annual runoff volume from the MMSD system. This, however, does not constitute a true cost-benefit curve, because it cannot be extrapolated to situations or locations not included in the study.

3A.9.7 Summary

While implementation of this technology has the benefit of reducing conveyance and treatment costs, optimizing the collection system capacity and, in conjunction with BMPs, reducing the pollutant loadings to the waterways, its application is limited to the seven sites included in the original study. Any additional locations discovered in the future should be analyzed in similar fashion.

3A.10 Other Technologies

The technologies listed in this section do not have production functions. They are technologies that are implemented at a single fixed level of performance and do not have a range of implementation.

The consideration for the use of these technologies in the 2020 FP and the RWQMPU will depend upon the outcome of the water quality modeling. If the water quality modeling indicates that the water quality indicator (pollutants) reduced by these technologies are at levels that are causing water quality standards or guidelines to be exceeded, the analysis of these technologies will be evaluated. If the indicators that these technologies reduce or control are not shown to be parameters of concern, the technologies will be not be used in the 2020 FP or RWQMPU alternatives analysis.

3A.10.1 Total Suspended Solids Indicator

Wastewater Treatment Plant – Conventional Final Effluent Filtration

Total suspended solids removal by conventional filtration of the final effluent was evaluated at JIWWTP and SSWWTP at the peak capacities for each plant. The technology assumes that the TSS concentration is reduced from 10 mg/L to 3 mg/L at JIWWTP and from 8 mg/L to 3 mg/L at SSWWTP.

The cost equations are based on a USEPA guide and then scaled to the ENR-CCI = 10,000 (2007) values. The total costs include the construction cost plus contingencies, non-construction costs, and the O&M cost for 20 years of operation.

This technology can remove 70% of the TSS load at JIWWTP and 63% of the load at SSWWTP. The total present worth cost of conventional treatment is $103 million at JIWWTP and $73 million at SSWWTP; the unit costs are $49 and $36 per pound of TSS removed.(26)

Wastewater Treatment Plant – Membrane Effluent Filtration

Membrane filtration provided a higher degree of TSS removal than conventional filtration. Conventional filtration, discussed above, is a necessary preliminary stage of filtration that must be implemented before membrane filtration can be applied. With membrane filtration, the TSS concentration can be reduced another five orders of magnitude from the concentration of conventional filtration, from 3 mg/L to 0.03μg/L. With membrane filtration 100% of the TSS load can be removed.

The cost of membrane filtration is approximately twice the costs of conventional filtration. Because conventional filtration is a necessary component of a membrane filtration system, the costs in Table 3A-18 are the combined cost of both conventional filtration and membrane filtration. The unit costs for membrane filtration are $114 per pound removed at JIWWTP and $87 per pound at SSWWTP.(27,28)

TABLE 3A-18
TOTAL SUSPENDED SOLIDS INDICATOR: TREATMENT PLANT EFFLUENT FILTRATION

	JIWWTP		SSWWTP	
	Conventional Filtration	**Membrane Filtration**	**Conventional Filtration**	**Membrane Filtration**
Design Average Daily Flow (MGD)	110	110	100	100
Design Peak Hourly Flow (MGD)	330	330	300	300
Total Present Worth Cost ($ Millions)	**$98**	**$344**	**$70**	**$286**
Unit Cost ($/lb removed)	$49	$114	$36	$87
Initial TSS (mg/L)	10	3	8	3
Filtered Effluent TSS (mg/L)	3	0.00003	3	0.00003
TSS Load Removed (tons/year)	1050	1510	1030	1640
TSS Load Removed (% of initial load)	70%	100%	63%	100%

JIWWWTP = Jones Island Wastewater Treatment Plant
MGD = Million Gallons per Day
TSS = Total Suspended Solids
lb = Pounds
SSWWTP = South Shore Wastewater Treatment Plant

Note:
All costs are estimated at an ENR-CCI of 10,000 (projected to be 2007).

3A.10.2 Coliform Indicator

Coliform can be reduced by filtration and disinfection. Three technologies were identified to reduce coliform: end of pipe disinfection of stormwater, CSO and SSO outfalls; ultraviolet disinfection of treatment plant effluent and membrane filtration of treatment plant effluent. The membrane filtration technology, which was presented for TSS, is also included here because of the benefits of the technology for coliform removal.

End of Pipe Disinfection – Stormwater/Sanitary Sewer Overflow(SSO)/Combined Sewer Overflow (CSO)

A conceptual satellite treatment plant near the CT-5/6 dropshaft that could treat combined sewer flow before being discharged into the Menomonee River near 25th Street as a CSO was reviewed in Section 3A.3.4. End of pipe disinfection was included in the conceptual treatment plant and the same issues discussed in Section 3A.3.4 apply to end of pipe disinfection as well. A satellite disinfection system would need a very high capacity to treat the peak flow rate. In addition, the disinfection system would only operate during the peaks of large storms; most of the time the system would not be operating. There are difficulties with maintaining and operating a disinfection system that is normally dry and must rapidly come on-line for a few hours during the peak of a storm. The same issues that apply to CT-5/6 are applicable to most CSO and SSO locations throughout the MMSD service area. Therefore, it was determined that end of pipe disinfection of CSO and SSO outfalls was not feasible.

Stormwater discharge from storm sewer outfalls was considered a nonpoint source. Therefore, disinfection of stormwater runoff is reviewed in Chapter 4, *Nonpoint Source Technology Analysis*.

Wastewater Treatment Plant - UV Disinfection

Ultraviolet (UV) disinfection uses ultraviolet light to destroy microorganisms to meet effluent E. coli limits. A UV system can treat the same capacity as a chlorination/dechlorination disinfection system in a smaller footprint. Though both UV and chlorination are effective at treating bacteria, UV is more effective at treating viruses and protozoa, such as cryptosporidium. Disinfection would be achieved by adding multiple UV lamps in various configurations within the existing chlorine contact basins and would be designed to achieve total fecal coliform of 100 #/mL or less.

Design Factors

Design factors that are important for denitrification are the UV transmittance, TSS concentrations, iron salts concentrations, the hardness level, and fecal coliform levels upstream of the UV system.

Effectiveness and Assumptions

A 3 log inactivation of fecal coliform would be required to meet WPDES permit requirement of 400#/mL; a 3.65 log inactivation of fecal coliform would be required to meet the current permit limit of 100#/mL fecal coliform.

The following limits must be met for the design factors listed above: UV transmittance needs to be between 65-70% or greater; TSS must be less than 50 mg/L, with solids no bigger than 50 microns; iron salts concentrations must be less than 4% of TSS, with total iron less than 0.3 mg/L; and the hardness level must be low enough to prevent UV lamp fouling – there is no specific limit, but larger concentrations create UV lamp fouling.

Cost

The present worth costs to implement UV disinfection at JIWWTP and SSWWTP at current capacity and an additional 50 MGD of capacity are shown in Tables 3A-19 and 3A-20 below. In both tables, the costs for the UV disinfection system at JIWWTP include the cost to build on piles.

TABLE 3A-19
COLIFORM INDICATOR: TREATMENT PLANT UV DISINFECTION FOR EXISTING 2000 CAPACITY

	Jones Island Wastewater Treatment Plant	South Shore Wastewater Treatment Plant
Design Average Daily Flow (MGD)	100	100
Design Peak Hourly Flow (MGD)	330	300
Total Present Worth Cost ($ Millions)	$19.8	$18.8

TABLE 3A-20
COLIFORM INDICATOR: TREATMENT PLANT UV DISINFECTION FOR ADDITIONAL 50% CAPACITY

	Jones Island Wastewater Treatment Plant	South Shore Wastewater Treatment Plant
Design Average Daily Flow (MGD)	50	50
Design Peak Hourly Flow (MGD)	150	150
Total Present Worth Cost ($ Millions)	$10.0	$10.0

Wastewater Treatment Plant Membrane Effluent Filtration as Disinfection

Membrane filtration benefits both the TSS indicator and the coliform indicator. The membrane filtration technology removes bacteria, protozoa, cryptosporidium, and most viruses. The present worth costs to implement membrane filtration at JIWWTP and SSWWTP at existing capacity are shown in Table 3A-21.

TABLE 3A-21
COLIFORM INDICATOR: TREATMENT PLANT MEMBRANE FILTRATION

	Jones Island Wastewater Treatment Plant	South Shore Wastewater Treatment Plant
Design Average Daily Flow (MGD)	100	100
Design Peak Hourly Flow (MGD)	330	300
Total Present Worth Cost ($ Millions)	$344	$286

MGD = Million Gallons per Day

Note:
All costs are estimated at an ENR-CCI of 10,000 (projected to be 2007).

3A.10.3 Nitrogen Indicator

Wastewater Treatment Plant - Denitrification

Biological denitrification is the microbial reduction of nitrate to nitrite and ultimately to nitrogen gas. Anoxic conditions, i.e., the absence of oxygen but the presence of nitrate, are required for denitrification to occur. Denitrification would be achieved by adding anoxic basins to the existing nitrifying activated sludge plant and would be designed to achieve total effluent nitrogen of 3 mg/L or less.

Design Factors

Design factors that are important for denitrification are the type of carbon substrate, nitrate concentration, reactor dissolved oxygen concentration, alkalinity, pH, and temperature.

Effectiveness and Assumptions

Average total nitrogen removal is 91%, based on an average influent concentration of 35 mg/L and effluent of 3 mg/L. The average annual amount of nitrogen removed at JIWWTP would be 4,800 tons/year, and at SSWWTP 6,600 tons/year.

A supplementary carbon source may be needed to achieve denitrification.

Cost

To implement denitrification at JIWWTP, the total present worth cost is $195 million for a capacity of 110 MGD design average daily flow. The cost assumes that all structures at JIWWTP would be constructed on piles.

At SSWWTP the cost is $161 million for a capacity of 100 MGD design average daily flow. These costs are based upon data from two reports.(29,30)

Wastewater Treatment Plant – Nitrification

Nitrification is the sequential oxidation of ammonium-nitrogen to nitrite-nitrogen and then nitrate-nitrogen. While it can be achieved at a treatment plant by means of several processes, it has been demonstrated that it can be achieved at JIWWTP and SSWWTP using the single-stage activated sludge process. However, this process requires a longer solids retention time and more air and aeration tank volume than a comparable non-nitrifying activated sludge process.

Design Factors

Solids retention time, influent and effluent nitrogen concentrations, reactor disolved oxygen (DO) concentration, aeration basin loading (volume), and sedimentation tank surface overflow rate are all important design factors in the nitrification process.

Experience

SSWWTP has been successful in achieving seasonal nitrification. JIWWTP has demonstrated the capability to achieve year round nitrification.

Effectiveness and Assumptions

The analysis assumes that the average concentration of influent ammonia of 7.8 mg/L is fully removed to 0 mg/L in the effluent. At JIWWTP, a nitrification process could remove 1,200 tons of ammonia (NH_3) per year. At SSWWTP, 1,600 tons per year can be removed.

Caveats

Nitrification can result in a mixed liquor that settles poorly. Nitrification can reduce the organic nitrogen content of waste solids and of Milorganite®.

Cost

The cost of a nitrification system is the additional cost of expanding the activated sludge process to include nitrification. At JIWWTP the total present worth cost is $492 million for a capacity of 110 MGD design average daily flow.

At SSWWTP the cost is $281 M for 100 MGD of capacity design average daily flow.(31)

3A.10.4 Phosphorus Indicator

Wastewater Treatment Plant – Biological Phosphorus Removal

Biological phosphorus removal (BPR) includes the microbial uptake of phosphorus by phosphorus storing bacteria that are then removed in the waste sludge. Though initially identified in Chapter 2, the BPR technology was not chosen for further analysis. Review of this technology indicated that biological removal of phosphorus is more efficient when nitrate has been reduced from the wastewater as is done in the denitrification process before being introduced to the phosphorus removal stage. Therefore, this technology is reviewed as part of the biological nutrient reduction (BNR) technology analysis.

Wastewater Treatment Plant – Biological Nutrient Reduction

The BNR technology includes both the microbial reduction of nitrate to nitrite and ultimately to nitrogen gas described under *Wastewater Treatment Plant – Denitrification* in Section 3A.10.3 and the microbial uptake of phosphorus by phosphorus storing bacteria that are then removed in the waste sludge. Anoxic conditions, i.e., the absence of oxygen but the presence of nitrate, are required for denitrification to occur. Anaerobic conditions, i.e., the absence of oxygen as well as nitrate, are required to encourage the uptake of phosphorus by phosphorus storing bacteria. BNR would be achieved by adding anoxic and anaerobic basins to the existing nitrifying activated sludge plant and would be designed to achieve total effluent nitrogen of 3 mg/L or less and total effluent phosphorus of 1 mg/L or less.

Design Factors

Design factors that are important for BNR are the type of carbon substrate, nitrate concentration, reactor dissolved oxygen concentration, alkalinity, pH, temperature, food to microorganism ratio, solids retention time, mixed liquor suspended solids, hydraulic retention time, percent return activated sludge, and percent internal recycle.

Effectiveness and Assumptions

Typically, a fully aerobic nitrification process will increase energy costs by 50% for the treatment of domestic wastewaters, compared to carbonaceous BOD removal only. Incorporation of an anoxic zone ahead of the aerobic zone in such a system will result in a reduction of the total aeration energy costs. Anaerobic and/or anoxic zones placed ahead of aerobic zones help discourage the growth of filamentous microorganisms in activated sludge and generally improve the sludge settling properties.

Caveats

BNR is more sensitive than conventional treatment. If a waste does not contain enough carbon, supplemental carbon may need to be added to the anaerobic zone to achieve the required nutrient removal. In addition, during periods of instability low levels of chemical addition may be required to meet P limits.

Cost

The unit costs for a BNR system are shown in Table 3A-22.

TABLE 3A-22

BIOLOGICAL NUTRIENT REDUCTION UNIT COST DATA

DADF, MGD	$/lb Nutrients (TN, TP) Removed
50	11.64
100	10.76
167	10.61

DADF = Daily Average Dry Weather Flow
MGD = Million Gallons per Day
lb = Pounds
TN = Total Nitrogen
TP = Total Phosphorus

Sources: Joint Committee of ASCE/WEF, Design of Municipal Wastewater Treatment Plants, 4th ed., ASCE Manuals and Reports on Engineering Practice No. 76 (Reston: ASCE Publications, 1988);
U.S. Environmental Protection Agency, Manual: Nitrogen Control (Washington, D.C.: U.S. Environmental Protection Agency, September 1993) EPA Publication No. 625R93010;
National Research Council, Managing Wastewater in Coastal Urban Areas (1993).

Wastewater Treatment Plant - Chemical Phosphorous Removal

Chemical phosphorus removal is a process in which metal salts of aluminum or iron are added to wastewater to react with phosphates to form insoluble aluminum or iron phosphate precipitates.

Design Factors

The design factors are the selection of metal salt, point of addition, and the required chemical dosage. The design of the chemical storage, handling and feed system is also an important part of the overall project design.

Experience

Chemical phosphorus removal is a proven process locally and nationwide.

Effectiveness and Drawbacks

80-95% of total phosphorus (TP) removal is achievable. Average effluent TP of <1 mg/L can be achieved if TSS<15 mg/L.

Chemical addition produces greater quantities of solids for disposal.

The costs for chemical phosphorous removal are shown in Table 3A-23.

TABLE 3A-23
CHEMICAL PHOSPHORUS REMOVAL UNIT COST DATA

DADF, MGD	$/lb P Removed
50	1.71
100	1.71
167	1.71

DADF = Daily Average Dry Weather Flow
MGD = Million Gallons per Day
lb = Pounds
P = Phosphorus

Sources: Joint Committee of ASCE/WEF, Design of Municipal Wastewater Treatment Plants, 4th ed., ASCE Manuals and Reports on Engineering Practice No. 76 (Reston: ASCE Publications, 1988);
U.S. Environmental Protection Agency, Design Manual: Phosphorus Removal (Washington, D.C.: U.S. Environmental Protection Agency, 1987) EPA Publication No. 625187001

References

(1) Brown and Caldwell, *Port Washington Road Relief Sewer Project, Alternatives Analysis, Draft Memorandum,* prepared for MMSD (Milwaukee: Brown and Caldwell, 2004)

(2) Ibid.

(3) Ibid.

(4) Ibid.

(5) Camp, Dresser and McKee, et al., *Indianapolis CSO Long Term Control Plan*, *Draft Report* (Cambridge: Camp, Dresser and McKee, 2001), Table 4-8

(6) Milwaukee Water Pollution Abatement Program, Combined Sewer Overflow Facility Plan Element, Executive Summary (June 1, 1980)

(7) Ibid.

(8) Ibid.

(9) Jim Ibach, E-mail from Jim Ibach of WES (April 13, 2005)

(10) Rod Pope, E-mail to Brown and Caldwell (February 14, 2005)

(11) Camp, Dresser and McKee, *Ad Hoc Question 29 – Final Report on Evaluation of Stormwater Reduction Practices in the Separate Sewer Area*, Memorandum to MMSD by Eric Loucks (March 9, 2004)

(12) Stormtech, Inc., *Evaluation of Stormwater Reduction Practices*, Memorandum to MMSD (March 1, 2003)

(13) CH2M Hill, Brown and Caldwell, Mithun, DRAFT Technical Memorandum, *Brightwater Treatment Plant Phase III Detail Design* (November 18, 2005)

(14) 99-WWF-8: Reducing Peak Rainfall-Derived Infiltration/Inflow Rates – Case Studies and Protocol (2003 study)

(15) Brown and Caldwell, *Infiltration and Inflow Reduction Demonstration Project Report* (Milwaukee: Milwaukee Metropolitan Sewerage District, February 15, 2006)

(16) EarthTech, *Pilot Project Report – Regional Infiltration and Inflow Control Program, King County, Washington* (October 2004)

(17) Water Environment Research Foundation, *Reducing Peak Rainfall-Derived Infiltration/Inflow Rates – Case Studied and Protocols*, WERF Project 999-WWF-8

(18) Milwaukee Metropolitan Sewerage District, Combined Sewer Overflow Amendment 1 Vol.1, Executive Summary (July 1, 1981)

(19) Milwaukee Metropolitan Sewerage District, Combined Sewer Overflow Amendment 1 Vol.1, Executive Summary (July 1, 1981)

(20) Kevin Vander Tuig, Tetra Tech, (Personal Communication), Lansing, Michigan (January 2005)

(21) George D. Barnes, City of Atlanta, GA (Personal Communication), (October, 18, 2005)

(22) Bonestroo, Rosene, Anderlik & Associates, Inc., Work Plan - *Wet Weather Flow Volume and Peak Management Project, Combined Sewer Service Area*, Village of Shorewood, (January 30, 2003) (Proposal to MMSD for Contract M03011P01)

(23) Stormtech, Inc., Memorandum, *Evaluation of Stormwater Reduction Practices* (March 1, 2003) (under MMSD Contract W91004E03)

(24) James Wasley, Associate Professor of Architecture, University of Wisconsin-Milwaukee, Principal investigator for the project, UWM as a "Zero Discharge Zone," Contract M03011P03, Scope of Services and Deliverables (January 31, 2005)

(25) Rust/Harza, *Internal Inspection of the Inline Storage System* (Final Report) (Milwaukee: Milwaukee Metropolitan Sewerage District, 2006)

(26) Milwaukee Metropolitan Sewerage District, Combined Sewer Overflow Amendment 1 Vol.1, Executive Summary (July 1, 1981)

(27) Ibid.

(28) HNTB, *Dos Rios Water Recycling Center Disinfection System Evaluation and Performance Upgrade Recommendation Project*, (November 17, 2003)

(29) Joint Committee of ASCE/WEF, *Design of Municipal Wastewater Treatment Plants, 4th ed.*, *ASCE Manuals and Reports on Engineering Practice No. 76* (Reston: ASCE Publications, 1988)

(30) U.S. Environmental Protection Agency, *Manual: Nitrogen Control* (Washington, D.C.: U.S. Environmental Protection Agency, 1993), EPA Publication No. 625R93010

(31) Joint Committee of ASCE/WEF, *Design of Municipal Wastewater Treatment Plants, 4th ed.*, *ASCE Manuals and Reports on Engineering Practice No. 76* (Reston: ASCE Publications, 1988)