UNIVERSITAT POLITÈCNICA DE VALÈNCIA

ETS INGENIERÍA DE CAMINOS, CANALES Y PUERTOS

FINAL MASTER THESIS

SUSTAINABLE DRAINAGE SYSTEMS (SUDS) IN INDUSTRIAL AREAS:
APPLICATION TO THE INDUSTRIAL AREA IN QUART DE POBLET (VALENCIA)

Presented by

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Summary

DOCUMENTO 0. MEMORIA

SISTEMAS DE DRENAJE SOSTENIBLE (SUDS) EN ÁREAS INDUSTRIALES:
APLICACIÓN AL ÁREA INDUSTRIAL DE QUART DE POBLET (VALENCIA)

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1 OBJETO

El documento presente se describe como el trabajo final de máster de la alumna Ana Álvarez Pérez con el fin de obtener el título en el Máster Universitario en Ingeniería de Caminos, Canales y Puertos en la Universidad Politécnica de Valencia (UPV), así como el diploma de ingeniero en l'École Nationale des Ponts et Chaussées (ENPC) en el marco del acuerdo del doble diploma realizado por la alumna.

El trabajo final de máster, de aquí en adelante llamado TFM, responde a la integración de SuDS en zonas industriales. Este trabajo se presenta como una solución a los sistemas de drenaje en zonas industriales en general, incluyendo su aplicación profesional al área industrial de Quart de Poblet, Valencia.

La tutoría del trabajo ha sido guiada por el doctor Ignacio Andrés Doménech, profesor titular en la Universidad Politécnica de Valencia, integrante del departamento de ingeniería hidráulica y medioambiente e investigador en el Instituto Universitario de Ingeniería del Agua y del Medio Ambiente (IIAMA). Además, el trabajo cuenta con el guiado de un cotutor, el doctor Damien Tedoldi, investigador especialista en hidrología urbana en l'Ecole Nationale des Ponts et Chaussées y profesor titular en l'Institut National des Sciences Appliquées en Lyon.

La organización del trabajo se presenta a continuación, dividiéndose en los siguientes documentos:

- DOCUMENTO Nº0: MEMORIA
- DOCUMENT Nº1: REPORT
    - Annex Nº1: Technical framework. Sustainable drainage systems
    - Annex Nº2: Initial data and design conditions
    - Annex Nº3: Solutions proposal
    - Annex Nº4: Comparative multi-criterial analysis
    - Annex Nº5: Development of the optimal solution
    - Annex Nº6: Monitoring and maintenance program
    - Annex Nº7: References
- DOCUMENT Nº2: MAPS
- DOCUMENT Nº3: ECONOMIC ASSESSMENT

2 ANTECEDENTES

La necesidad de implantación de sistemas de drenaje eficientes se hace cada vez más presente. Gran parte de esta necesidad viene determinado por el desarrollo de las ciudades y de las zonas urbanizadas. El desarrollo de las ciudades conforme avanza la sociedad implica un aumento de la impermeabilización del suelo, esta impermeabilización tiene efectos negativos directos e indirectos sobre diversos aspectos. La principal problemática la encontramos en la alteración de las capacidades naturales intrínsecas del suelo. Entre los aspectos negativos a los que afecta la alteración de las capacidades naturales del suelo podemos encontrar:
- Disminución de la capacidad de retención y filtración del agua pluvial
- Inertización y desertización de la superficie terrestre
- Aumento de la temperatura ambiente en las ciudades
- Deterioro de la calidad atmosférica
- Alteración paisajística
- Desnaturalización del entorno

La superpoblación de las ciudades queda plasmada en datos proporcionados por el Banco Mundial. Más de 4000 millones de habitantes en todo el mundo, que supone más del 50% de la población mundial, viven en ciudades. Esta tendencia de crecimiento continuará con unas perspectivas de duplicación de la población urbana en 2050, donde casi 7 de cada 10 personas vivirán en ciudades.

Este desarrollo de la polis afecta al calentamiento de la atmosfera, entre varios aspectos popularmente conocidos como la contaminación gaseosa de los vehículos se le añade, centrado en el tema que nos concierne, la reverberación del calor que se adhiere a superficies permeables como los pavimentos o el asfalto. Este fenómeno aumenta notablemente la temperatura ambiente; incluyendo que, en ciudades como Valencia, la temperatura de los pavimentos pueda alcanzar cerca de los 90 grados.

Sin embargo, la mayor consecuencia de la impermeabilización viene dada por la modificación del ciclo de recursos hídricos; donde un cambio provoca la modificación del camino natural del agua, provocando el aumento de:
- Volúmenes de escorrentía
- Caudales pico
- Incremento de caudales, siendo más bajos en épocas de sequía y más altos en épocas de lluvia
- Contaminación

El sistema convencional se centra en la recopilación del agua pluvial a través de las redes de saneamiento. Una metodología que intenta evacuar el agua lo más rápido posible llevándolo fuera de la zona de actuación y en su caso, ser tratada en estaciones de depuración. Un sistema como este, se enfrenta a una problemática cuando sus capacidades se ven inferiores a aquello requerido. Grandes cantidades de agua pluvial a ser tratadas significa vertidos de aguas contaminadas, reducción del rendimiento de las estaciones de depuración o incluso desastres como inundaciones locales. Se puede comprobar que hasta ahora los sistemas de drenaje tradicionales se han centrado más en la cantidad de agua a gestionar, obviando otros aspectos como la calidad o el propio servicio que se le puede dar a esta agua.

En este contexto, entran en plano los sistemas urbanos de drenaje sostenible (SuDS). Los SuDS llegan como la solución a una problemática que afecta a muchas áreas del mundo. La solución radica en un cambio de gestión del agua pluvial, en lugar de pretender actuar al final del proceso se trataría de intentar retener el tratamiento del aguan en el origen. Imponiéndose medidas exclusivamente hidrológicas (estudio, planificación y cuantificación del recurso hídrico) en lugar de hidráulicas (diseño y operación de obras civiles para el aprovechamiento de recurso).

Los SuDS se presentan como una solución con una gestión inteligente y eficaz del agua donde su principal objetivo no es recolectar el agua, encaminarla y tratarla si no intentar tratarla lo antes posible gracias a la infiltración. La retención descentralizada permite infiltrar y retener en el origen el agua para liberarla progresivamente en momentos pico y aprovecharla para usos de agua no potable tales como el regadío.

3 CONTEXTO NORMATIVO

El contexto normativo de los sistemas de drenaje, particularizándose en drenaje sostenible, viene parametrizado por los principios de gobernanza de la legislación europea y española, siendo estos la mejora del estado de las masas de agua, la protección frente inundaciones y sequías, la adaptación y mitigación al impacto del cambio climático, la reducción del consumo energético en el ciclo urbano del agua, la conservación de la biodiversidad y el refuerzo de los valores del agua y de los espacios verdes dedicados a la ciudadanía.

En este apartado se ha recopilado las normativas europeas, nacionales, autonómicas y municipales vigentes que rigen los sistemas de drenaje sostenible.

3.1 Normativa europea

- **Directiva 2000/60/CE** del Parlamento Europeo y del Consejo, de 23 de octubre de 2000, por la que se establece un marco comunitario de actuación en el ámbito de la política de aguas. Nace como respuesta a la necesidad de unificar las actuaciones en materia de gestión de agua en la Unión Europea, tomando medidas para proteger las masas de agua tanto en términos cualitativos como cuantitativos y garantizar así su sostenibilidad, y marcando unos plazos de implantación de la directiva en los diferentes Estados Miembros de la UE, así como de consecución de objetivos.

- **Directiva 2007/60/CE** del Parlamento Europeo y del Consejo, del 23 de octubre de 2007, relativa a la evaluación y gestión de los riesgos de inundación.

3.2 Normativa nacional

- **Real Decreto 1/2016**, de 8 de enero, por el que se aprueba la revisión de los Planes Hidrológicos de las demarcaciones hidrográficas del Cantábrico Occidental, Guadalquivir, Ceuta, Melilla, Segura y Júcar, y de la parte española de las demarcaciones hidrográficas del Cantábrico Oriental, Miño – Sil, Duero, Tajo, Guadiana y Ebro. En su artículo 44 se dice: "Las nuevas urbanizaciones, polígonos industriales y desarrollos urbanísticos que puedan producir alteraciones en el drenaje de la cuenca o cuencas interceptadas deberán introducir sistemas de drenaje sostenible (uso de pavimentos permeables, tanques o dispositivos de tormenta, etc.…) que garanticen que el eventual aumento de escorrentía respecto del valor correspondiente a la situación preexistente puede ser compensado o es irrelevante".

- **Real Decreto 638/2016**, de 9 de diciembre, por el que se modifica el Reglamento del Dominio Público Hidráulico aprobado por el Real Decreto 849/1986, de 11 de abril, el Reglamento de Planificación Hidrológica, aprobado por el Real Decreto 907/2007, de 6 de julio, y otros reglamentos en materia de gestión de riesgos de inundación, caudales ecológicos, reservas hidrológicas y vertido de aguas residuales. En su artículo 126 ter se especifica: "Las nuevas urbanizaciones, polígonos industriales y desarrollos urbanísticos en general, deberán introducir sistemas de drenaje sostenible, tales como superficies y acabados permeables, de forma que el eventual incremento del riesgo de inundación se mitigue. A tal efecto, el expediente del desarrollo urbanístico deberá incluir un estudio hidrológico-hidráulico que lo justifique".

3.3 Normativa autonómica

- **Decreto 201/2015**, de 29 de octubre, del Consell, aprobó el Plan de Acción Territorial sobre Prevención de Riesgo de Inundación en la Comunidad Valenciana (PATRICOVA). Este Plan defiende la gestión de la Infraestructura Verde frente a las inundaciones. Se define la Infraestructura verde al sistema territorial básico formado por los espacios de mayor valor ambiental, cultural, paisajístico y visual, las áreas críticas del territorio por su susceptibilidad a los riesgos, así como sus conexiones ecológicas y funcionales. Aunque se considere un plan de prevención frente a inundaciones consecuencia del desbordamiento de cauces o subida del nivel freático, incorpora la recomendación de implantación de SUDS por su contribución, entre otras, en la reducción de daños por lluvias de corto período de retorno. En el Titulo IV "De las Actuaciones de Defensa", Artículo 23, se dice que "En el diseño de la Infraestructura Verde, se fomentará el uso de Sistemas Urbanos de Drenaje Sostenible". Además, se añade que "se fomentará el uso de Sistemas Urbanos de Drenaje Sostenible en todos los municipios de la Comunidad Valenciana".

- **Resolución 997/IX**, de 30 de mayo de 2017, por la Presidencia de Les Corts de la Generalitat Valenciana, sobre la Incorporación de medidas de prevención y técnicas relacionadas con el uso de SUDS en la infraestructura verde incluye: "En cumplimiento de lo dispuesto por el artículo 95.1 del Reglamento de Les Corts, se ordena publicar en el Butlletí Oficial de les Corts la Resolución 997/IX, sobre la incorporación de medidas de prevención y técnicas relacionadas con el uso de sistemas de drenaje sostenible en el diseño de la infraestructura verde incluida o asociada a los planes de ordenación del territorio, aprobada por la Comisión de Obras Públicas, Infraestructuras y Transportes en la reunión del 30 de mayo de 2017".

3.4 Normativa municipal

- **Ordenanza de Protección del Medio Ambiente relativa al Alcantarillado y Vertidos a la Red Municipal**, publicada por el Ayuntamiento de Quart de Poblet, aprobada con carácter definitivo el 29 de diciembre de 2016, incluyendo en el capítulo 2, punto 12 "El vertido de estas aguas limpias deberá promover soluciones ambientales sostenibles y de bajo impacto, como su derivación a cauces naturales o infiltración pasiva en zonas despejadas".

- **Estrategia de Desarrollo Urbano Sostenible Integrado (EDUSI)** de Quart de Poblet, cofinanciada mediante el Programa Operativo FEDER de crecimiento sostenible 2014- 2020, convocadas por Orden HAP/2427/2015 de 13 de noviembre como una actuación encaminada a la mejora y rehabilitación del entorno urbano

4 ALCANCE DEL TRABAJO

El presente trabajo tiene dos objetivos. El primero es el de determinar la implementación de sistema de drenajes sostenibles que se pueden integrar en zonas industriales, identificando de manera justificada aquellas tipologías que mejor se adecuen a este tipo de entorno. Siendo los sistemas de drenaje sostenibles innovadores de por sí, su aplicación en áreas industriales no ha sido estudiada con detalle y pocos documentos hacen referencia a los beneficios de estos sistemas en las áreas industriales donde la impermeabilización y el contenido en contaminantes hacen que el estado de las aguas de escorrentía sea muy perjudiciales para el medio receptor, ya sea en cantidad como en calidad.

En segundo lugar, se determinará la solución óptima para tratar de manera eficaz el agua pluvial en el polígono industrial de Quart de Poblet, Valencia, España teniendo en cuenta criterios financieros, hidráulicos, medioambientales, energéticos y sociales. Atendiendo a este objetivo se buscará conseguir que, mediante el sistema de drenaje, se consiga verter al medio receptor el mismo caudal que en estado natural, previa la urbanización del polígono industrial. Para la consecución de este objetivo, se realizará un análisis de los condicionantes físicos y administrativos de la zona de estudio y sus parámetros de diseño para determinar los sistemas de drenaje que mejor se adapten y resuelvan la problemática en el área industrial.

A través de guías de recomendaciones, se realizará un dimensionamiento de los sistemas de drenaje ateniéndose cada uno a sus especificaciones. Posteriormente se realizará una verificación con el programa SWMM que permitirá comprobar que cada solución está bien dimensionada a partir de las variables hidráulicas determinadas y finalmente se determinará la solución óptima a partir de un análisis multicriterio mediante la herramienta E2Stormed.

De este modo, este trabajo no solo indicará los sistemas de drenaje a implantar que mejor solución la problemática de la aplicación, si no que analizará los múltiples beneficios de los SuDS, concretizando en las áreas industriales, y presentando la necesidad de encuadramiento normativo e impulso por parte de organismos públicos y privados para invertir más recursos en estos sistemas.

5 LOCALIZACIÓN

El polígono industrial al que hace referencia este estudio se localiza en Quart de Poblet, municipio de la Comunidad Valencia, España y perteneciente a la provincia de Valencia.

Situado entre la Huerta Oeste valenciana, la superficie del término es cruzada por el río Turia por el norte. Por el oeste circula la rambla del Poyo, curso de agua estacional cuya cuenca se encuentra entre el río Turia y Júcar y la del Barranco de Picassent.

Figura 1. Localización a nivel estatal. Fuente: Visor cartográfico GVA

Figura 2. Localización a nivel comarcar. Fuente: Visor cartográfico GVA

Figura 3. Localización a nivel municipal. Fuente: Visor cartográfico GVA

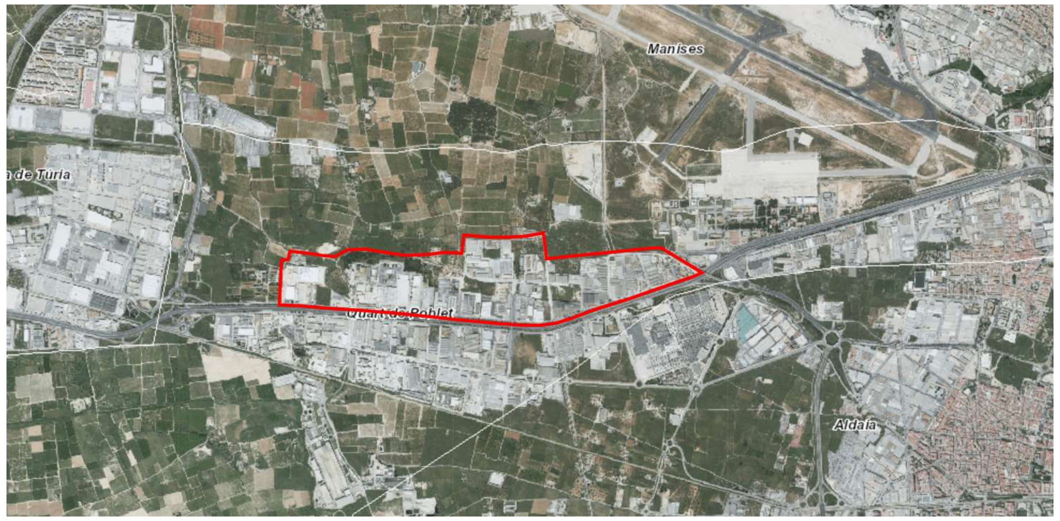

Figura 4. Localización dentro de Quart de Poblet. Fuente: Visor cartográfico GVA

Los principales ejes de tráfico rodado situados al oeste del área metropolitana de Valencia atraviesan el término municipal de Quart de Poblet, por lo que lleva a considerar que Quart de Poblet está muy bien integrado a la red de infraestructuras de transporte del área metropolitana valenciana.

Así, tanto el casco urbano como los numerosos polígonos industriales están bien conectados por autovía a los municipios circundantes y otras carreteras de gran capacidad, facilitando el transporte por carretera, además de las conexiones del transporte público ferroviario.

Los principales ejes de tráfico que se presentan en la zona de estudio son:

- Autovía A-3/E-901 Madrid-Valencia, discurre longitudinalmente al sur del municipio y proporciona tres accesos
- Autovía V-30 de circunvalación, situada en paralelo al cauce del río Turia, atraviesa de forma diagonal desde el noroeste hasta el sureste
- Autovía V-11, conexión entre Quart de Poblet y el aeropuerto de Valencia.
- Carreteras CV-408 y CV-31, carreteras locales y provinciales que sirven de nexo con núcleos vecinos pasando por las instalaciones industriales
- Líneas de metro 3, 5 y 9 que fueron adaptadas para sustituir a la antigua estación de ferrocarril de la línea C-4 Valencia-Norte - Ribarroja del Turia.
- Líneas de metrobus que conectan Quart de Poblet con Valencia y otras localidades; 106 (Torrent), 150 (Aeropuerto de Manises), 161 (Valencia).

6 EMPLAZAMIENTO Y SITUACIÓN ACTUAL

6.1 Cuenca del Poyo

La cuenca en la que se localiza la zona del proyecto se denomina cuenca del Poyo y está conformada por los barrancos Poyo, Pozalet y Saleta. Esta cuenca está situada entre los ríos Turia y Júcar y tiene una extensión de 450 Km2.

La problemática relacionada con las precipitaciones de elevada densidad, añadidas a el aumento considerable del desarrollo urbanístico de las zonas urbanas de la cuenca, ha provocado que los barrancos contengan caudales muy elevados a la entrada de las zonas urbanas provocando riesgo de inundación de varios términos municipales. Además, siendo el lago de la Albufera el punto final actual de las aguas, se añade el riesgo de aportación de sedimentos a la Albufera.

Figura 5. Localización de la cuenca del Poyo. Fuente: F. Franch et al., 2010

Aproximándose en mayor medida en la zona de estudio, el polígono industrial de Quart de Poblet se sitúa aguas abajo del Barranco del Pozalet y aguas arriba del Barranco de la Saleta. El punto más débil de la zona se sitúa entre Loriguilla y el by-pass de la autovía A7, con una capacidad de desagüe muy inferior a la del propio cauce. Entre Loriguilla y la autovía A3, el cauce deja de estar definido provocando un área de laminación donde las aguas discurren sin control afectando a los polígonos colindantes, siendo el polígono del estudio uno de ellos.

Aunque el Barranco del Poyo no sea un punto de salida del agua del polígono industrial, es importante conocer su comportamiento hidrográfico debido a que, en situación de lluvias de cierta magnitud, parte de los caudales desbordados del barranco del Poyo, aguas abajo de la A7, se dirigen hacia el norte aumentando los caudales del Pozalet y consecuentemente de La Saleta.

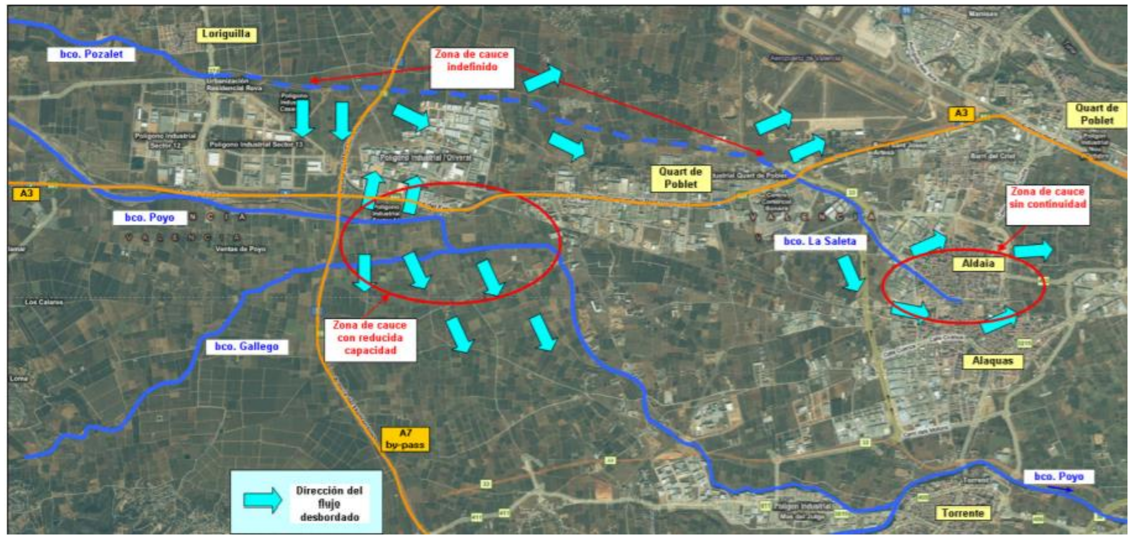

Figura 6. Comportamiento hidráulico de los barrancos Pozalet-Saleta. Fuente: F. Franch et al., 2010

Actualmente, el barranco de La Saleta presenta en su tramo inicial una capacidad suficiente para transportar un caudal en torno a los 100 m3/s. Esta capacidad va disminuyendo progresivamente hasta anularse totalmente en pleno casco urbano de Aldaia, lo que sitúa a esta localidad como la más afectada en cuanto a riesgo de inundación se refiere. El barranco Pozalet complica aún más la situación de las poblaciones vinculadas a La Saleta al ser La Saleta receptor de las aguas de Pozalet, lo que hace que el caudal pico en situación de avenidas aumente considerablemente. Un ejemplo de situación de desbordamientos, que resultó catastrófica, fue la avenida del año 2000 con un periodo de retorno de 500 años.

En la siguiente imagen, se ha obtenido un resultado de la simulación de inundaciones para un periodo de retorno de 500 años, y así poder plasmar las consecuencias visualmente.

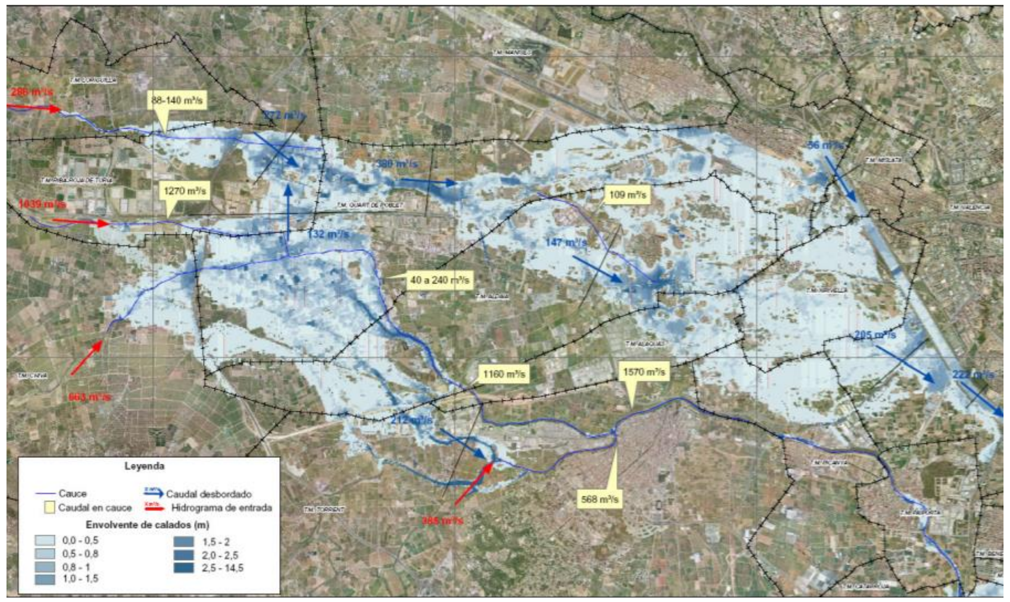
Figura 7. Simulación de inundación con evento representativo de 500 años de periodo de retorno (avenida año 2000). Fuente: F. Franch et al., 2010

La confederación Hidrográfica del Júcar ha presentado diferentes soluciones para evitar el problema de desbordamiento de cauce e inundaciones. Una de ellas es el planteamiento de la ejecución de una conducción cerrada a través del polígono industrial de la zona norte de Aldaia, derivándose una parte por el cunetón de la carretera CV-33 y creando una conexión entre el Barranco de La Saleta y el río Turia.

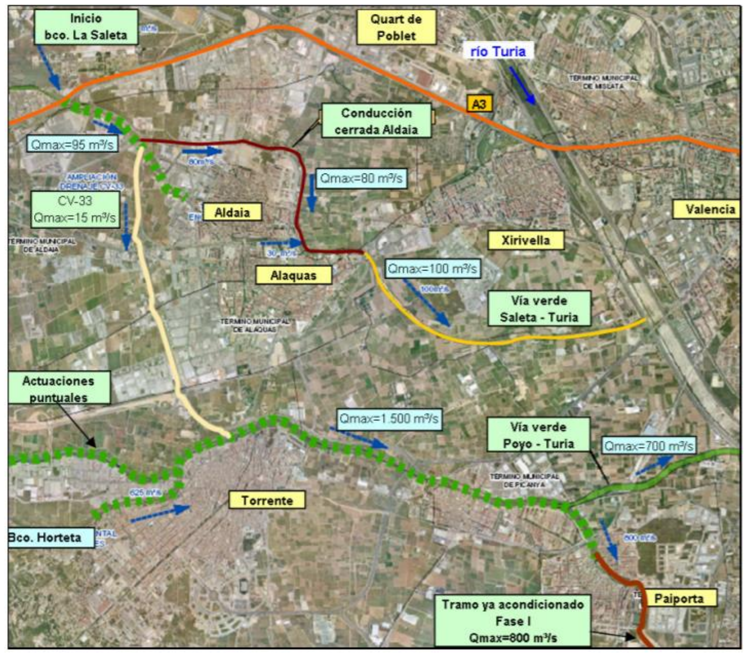

Figura 8. Solución de la CHJ para el barranco de La Saleta. Fuente: F. Franch et al., 2010

6.2 Polígono industrial en Quart de Poblet

Este trabajo, busca solucionar la problemática a una menor escala, centrándose en el polígono industrial en Quart de Poblet y sus aportaciones al Barranco de la Saleta. Actualmente, el polígono industrial de estudio no posee red de drenaje por lo que toda la escorrentía del agua pluvial recorre de manera libre las calles divergiendo en el Barranco de la Saleta. Esta situación es preocupante para la Confederación Hidrográfica del Júcar, que ve comprometido el Barranco de la Saleta debido a la gran carga de agua que conduce. El desarrollo urbanístico de las zonas colindantes al Barranco ha provocado un aumento considerable de la escorrentía llevándolo a sus límites, favoreciendo el desbordamiento de los cauces y el riesgo de inundaciones de las poblaciones adyacentes.

Ante esta situación, la Confederación Hidrográfica del Júcar requiere al ayuntamiento de Quart de Poblet que solucione la problemática relacionada con la cantidad de agua vertida, incitándole no solo a que se construya un sistema de drenaje que consiga encauzar de manera controlada el agua de lluvia, sino que además consiga reducir la cantidad de agua vertida al Barranco de la Saleta. Idealmente, se busca el objetivo ambicioso de que se consiga verter al barranco de la Saleta el mismo caudal que se vertía previa a la urbanización del polígono industrial. Por ello, y siempre teniendo en cuenta que trabajamos sobre una zona industrial donde las limitaciones por contaminantes tienen un papel principal, se ha motivado a buscar otra alternativa a la posible red de drenaje convencional para poder laminar el agua de escorrentía y controlar los contaminantes vertidos.

Para hacerse a una idea de las características del polígono industrial, se recoge a continuación una serie de fotografías tomadas recientemente de la zona de estudio que afirman la falta de sistemas de drenaje en la zona y el estado de las carreteras del polígono industrial. Las fotografías corresponden a pocas horas después de un evento de precipitación pudiendo reconocer varias áreas donde el agua permanece estancada en superficie debido a la falta de sistema de drenaje.

Así mismo, las fotografías demuestran el estado primitivo de las carreteras, compuestas por caminos de tierra y en el mejor de los casos, asfaltado de hormigón y gravilla. Cabe destacar pequeñas excepciones, como la calle más al este del polígono donde se ha podido comprobar que han comenzado obras de acondicionamiento de las carreteras mediante asfaltado con ligantes bituminosos.


Figura 9. Estado de la carretera calle Polígono 13, cuenca N2


Figura 10. Estado de la carretera camí de la Canyada, cuenca N1


Figura 11. Estado de la carretera calle Polígono 13, cuenca N1


Figura 12. Estado de la carretera de la Pinadeta, cuenca N3


Figura 13. Estado de la zona verde en la calle Polígono 12, cuenca N1

Figura 14. Estado de la carretera calle Canal Xúquer-Turia, cuenca N3

Figura 15. Inicio de acondicionamiento de la calle Sequia de Mislata, cuenca N1

7 DATOS DE PARTIDA Y CONDICIONANTES DE DISEÑO

Se han determinado los aspectos y parámetros relevantes del ámbito de estudio tales como la ordenación urbanística, topografía, geología, hidrogeología, geotecnia, pluviometría y el estado natural de las cuencas previa urbanización, para conocer su comportamiento y los parámetros objetivos que ayudarán a diseñar las diferentes soluciones.

Todo lo mencionado a continuación se puede encontrar explicado de manera detallada y con mayor profundidad en el *Annex Nº2: Initial data and design conditions.*

7.1 Ordenación urbanística

La Ordenación urbanística y territorial de la zona de estudio se realiza mediante el Plan General de Ordenación Urbanística (PGOU) de Quart de Poblet, publicado en 2002. Partiendo del PGOU, que identifica las parcelas de uso privado y dotaciones públicas que corresponden a viario y zonas verdes, se procede a dividir la zona en tres usos principales del suelo correspondientes con la respuesta hidrológica de cada uno y la cantidad de escorrentía que generan.

Figura 16. División parcelas en uso de suelos, basado en el PGOU

Se divide así la zona de estudio en subcuencas de parcelas privadas en rosa, de viario en azul y de zonas verdes en verde.

7.2 Topografía

La topografía es un gran condicionante para el diseño de las posibles soluciones. La determinación de las pendientes del terreno es importante para poder determinar el curso natural de la escorrentía superficial y así conocer el volumen de agua que deberá tratarse en cada superficie y mejor diseñar el sistema de drenaje. Además, las pendientes suponen un factor determinante, debido a que ciertos sistemas de drenaje sostenible están limitados por la pendiente del terreno. El análisis de las parcelas residenciales privadas no entra en el objeto de estudio, aunque se tiene en cuenta para la generación de escorrentía.

En la siguiente imagen, se muestra el plano de las pendientes longitudinales de cada viario obtenidas tras el tratamiento de la información de los mapas cartográficos. Asimismo, se puede encontrar esta información a escala en el *Document Nº2: Maps.*

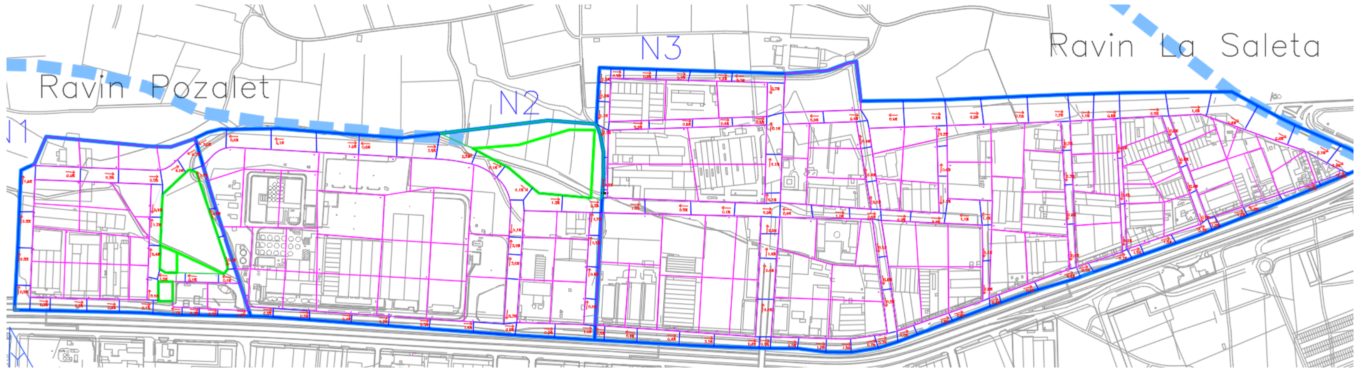

Figura 17. Pendientes longitudinales del área de estudio

7.3 Geología y geotecnia

La zona de estudio, que pertenece al eje sinclinal de la Cadena Ibérica, se forma durante el Mioceno como consecuencia del acortamiento del área ocupada por el mar mesozoico que produce la emersión de áreas de relieve positivo y áreas deprimidas que se alinean según ejes ibéricos. La región valenciana queda enclavada en una de estas depresiones, que es invadida por el mar en los tiempos del Terciario Inferior. Este mar depositó materiales arenosos, en principio conglomeráticos, formándose un golfo de dimensiones mayores al actual. En el Terciario Superior se produce una regresión del mar, creándose un área de aspecto lagunar, en la que se depositan las margas y calizas, que fue rellenándose lentamente por los aportes de los flujos de aguas procedentes de las zonas circundantes. Posteriormente, la generación de un clima más extremo que el actual y la red hidrológica dieron lugar a los depósitos cuaternarios procedentes de la erosión de las montañas aguas arriba, mientras que la línea de costa retrocedía, abandonando antiguos sedimentos litorales.

En lo que concierne la estratigrafía, la zona de estudio está localizada en los terrenos del periodo cuaternario, concretamente en los depósitos continentales correspondientes al pleistoceno inferior con costra calcárea y al pleistoceno superior con mantos de arroyada modernos, siendo arcillas arenosas rojas con cantos de costra.

La tectónica en el lugar del proyecto se presenta como una amplia depresión morfológica de origen tectónico complejo. Esta depresión, que recibe el nombre geográfico de Huerta de Valencia, se extiende entre el mar y las estribaciones de los relieves de Chiva y Buñol y está flanqueada hacia el norte por los relieves de Náquera, y al sur por los de Cullera-Alginet. La zona deprimida representa un eje sinclinal de la Cadena Ibérica, en el que se han depositado materiales claramente posteriores al momento principal de la compresión creadora de las estructuras «ibéricas». Por otra parte, esta depresión tectónica ha sido afectada por movimientos posteriores relacionados causalmente con la tectónica de las áreas «béticas» situadas más al Sur, que a su vez debe ser responsable de los movimientos más recientes de las costas mediterráneas.

7.4 Hidrogeología

7.4.1 Peligrosidad de inundación

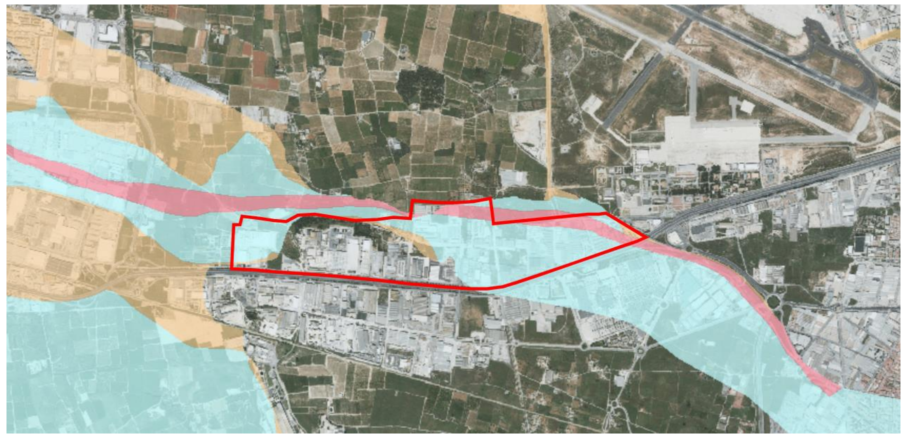

Figura 18. Mapa de peligrosidad de inundación por el PATRICOVA. Fuente: Visor cartográfico GVA

La zona norte del polígono está considerada una zona de peligrosidad de nivel 1, identificada en rosa en el mapa, con una frecuencia alta de periodo de retorno de 25 años y un calado que supera los 0,8 metros. La totalidad de la cuenca N3 y gran parte de la N2 está clasificada en zona de peligrosidad nivel 3, identificada en azul claro, con frecuencia alta de periodo de retorno 25 años y calado bajo, inferior a los 0,8 metros. En lo que se refiere a la peligrosidad geomorfológica, identificada en naranja, la zona de estudio se caracteriza por una pequeña zona de peligrosidad geomorfológica al este y en el centro del polígono industrial.

7.4.2 Riesgo de inundación

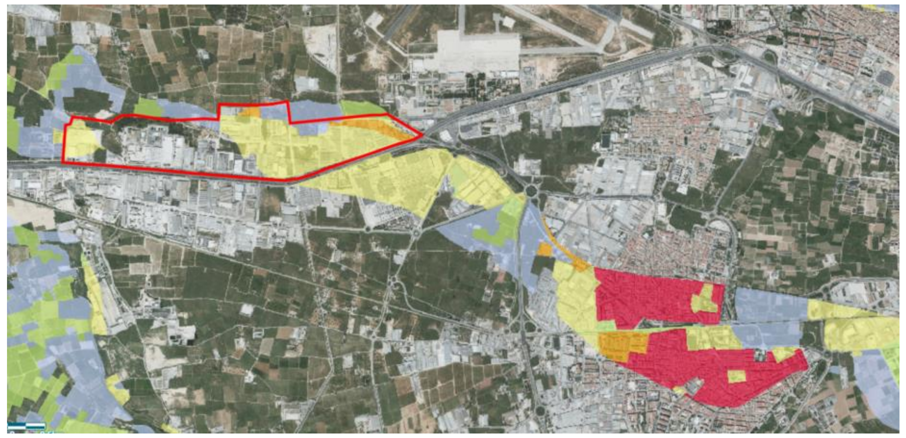

Figura 19. Mapa de riesgo de inundación por el PATRICOVA. Fuente: Visor cartográfico GVA

El plano de riesgo de inundación muestra que la zona de proyecto es una zona donde generalmente el riesgo es medio en amarillo, conteniendo algunas zonas de riesgo bajo en verde, y muy bajo en azul, y, en menor medida, zonas de riesgo alto en naranja. Como se indicó en la problemática sobre los vertidos del agua de escorrentía, la zona próxima al polígono industrial es el municipio de Aldaia con un riesgo de inundación muy alto, identificado en rojo.

7.5 Permeabilidad

Figura 20. Mapa de permeabilidades. Fuente: Visor cartográfico del GVA

Toda la zona de proyecto está clasificada como una zona de permeabilidad alta en verde claro, siendo colindante al este con una zona de permeabilidad muy alta, en verde oscuro. El coeficiente de permeabilidad del terreno considerado para la fase de diseño es:

$$Kc = \mathbf{12,9\ mm/h} = 3{,}58 \cdot 10^{-6}\ m/s$$

7.6 Periodo de retorno

El periodo de retorno de diseño considerado ha sido el indicado en el Plan de Acción Territorial sobre Prevención del Riesgo de Inundaciones de la Comunidad Valenciana (PATRICOVA, 2015). Correspondiente a:

$$\boldsymbol{Tr = 15\ años}$$

7.7 Análisis pluviométrico

El análisis pluviométrico se ha realizado a partir de los datos obtenido de la estación pluviométrica de Valencia – Aeropuerto, que recoge los datos de la serie histórica desde 1966 hasta 2019, correspondiendo a 54 años de recopilaciones de datos. Mediante estos datos, se ha realizado un análisis de frecuencia volumétrica y un análisis extremal de las precipitaciones máximas diarias para diferentes periodos de retorno y de este modo obtener las lluvias de diseño que servirán para el dimensionamiento y la modelización del caso de estudio.

7.7.1 Análisis frecuencia volumétrica

La precipitación media mensual y el número promedio de días de lluvia al mes para la serie disponible es la siguiente:

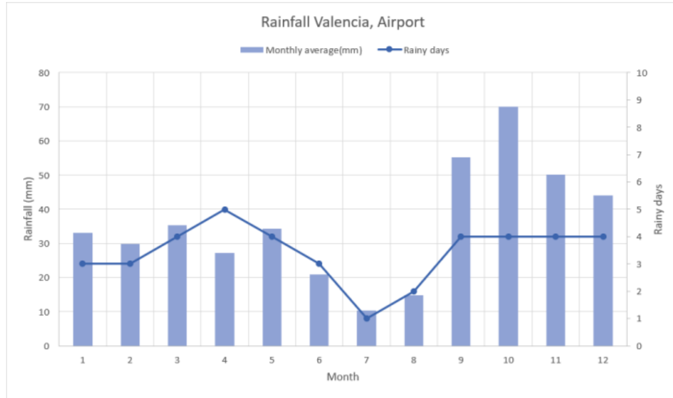

Figura 21. Precipitación media mensual y número promedio de días de lluvia al mes

Mediante los datos pluviométricos se puede identificar la función distribución de la precipitación diaria de muestra representa la probabilidad de no excedencia para cada volumen de precipitación diario, identificando los percentiles principales.

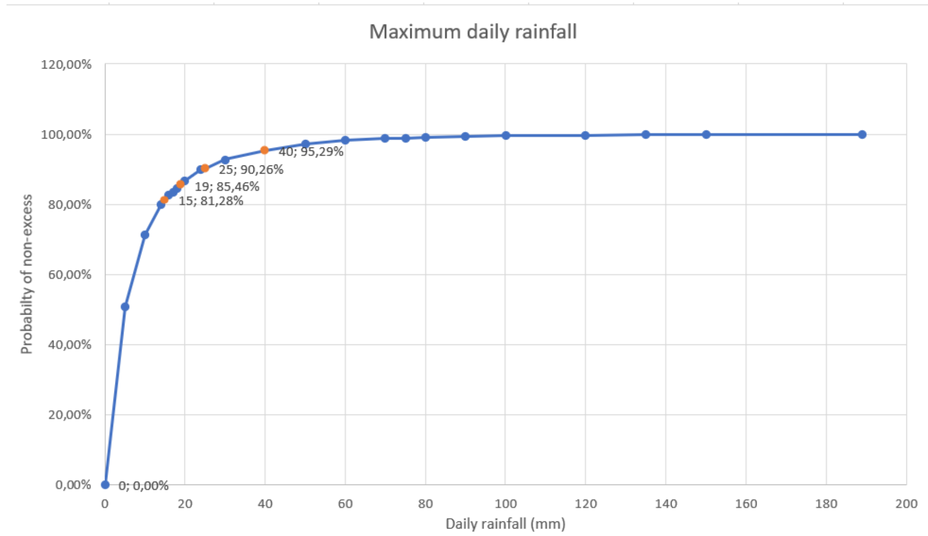

Figura 22. Representación gráfica de la probabilidad de no excedencia del valor de precipitación diaria

De los resultados se analiza:

- En el 80% de los días de lluvia, la precipitación no supera los 15 mm
- En el 85% de los días de lluvia, la precipitación no supera los 19 mm
- En el 90% de los días de lluvia, la precipitación no supera los 25 mm
- En el 95% de los días de lluvia, la precipitación no supera los 40 mm

7.7.2 Análisis extremal

Se determinan los eventos extremales para poder diseñar un sistema de drenaje que pueda funcionar de manera satisfactoria, sin producir inundaciones locales que pudiesen provocar daños materiales y medioambientales.

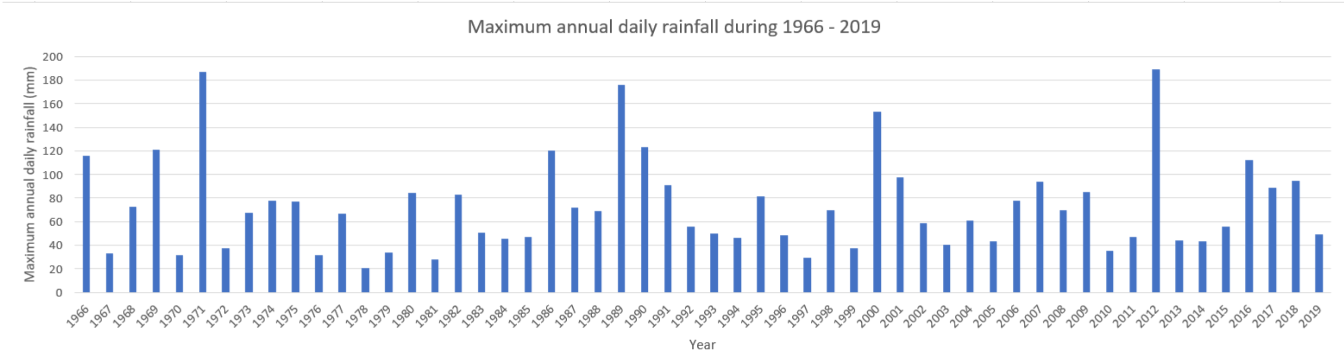

Figura 23. Máxima precipitación diaria durante el periodo 1966 - 2019

Mediante el análisis estadístico de la serie estudiada se ha podido identificar la distribución teórica que más se aproxima a la serie analizada a partir del método de la máxima verosimilitud, siendo esta la distribución SQRT-et-MAX cuya representación es la siguiente:

$$F_x(x) = e^{\left(-k*(1+\sqrt{\alpha x})*e^{(-\sqrt{\alpha x})}\right)}$$

Para los datos de la serie los valores de los parámetros son:

Tabla 1. Parámetros de la serie SQRT-et-MAX

Parámetro	Valor
k	26,2913
α	0,49285

Identificando así la precipitación diaria máxima de la función para cada periodo de retorno. Para el caso de estudio donde se ha determinado que el periodo de retorno de diseño será de 15 años obtenemos una precipitación máxima diaria de 135,02 mm.

Tabla 2. Precipitación máxima diaria para cada periodo de retorno

Periodo de retorno (años)	Precipitación máxima diaria (mm)
2	61,60
5	94,56
10	119,70
15	135,02
25	155,15
50	184,08

7.7.3 Chaparrones de diseño

Tras el análisis pluviométrico, se han determinado los chaparrones de diseño correspondientes a la curva IDF, que estiman la intensidad máxima de precipitación correspondiente al tiempo de concentración de la cuenca para un periodo de retorno determinado y los chaparrones de diseño correspondientes los registros de precipitación observada por el método de la tormenta Gamma de dos parámetros, conocido como G2P (García-Bartual y Andrés-Doménech, 2017).

La curva IDF determinada para un periodo de retorno de 15 años ha sido:

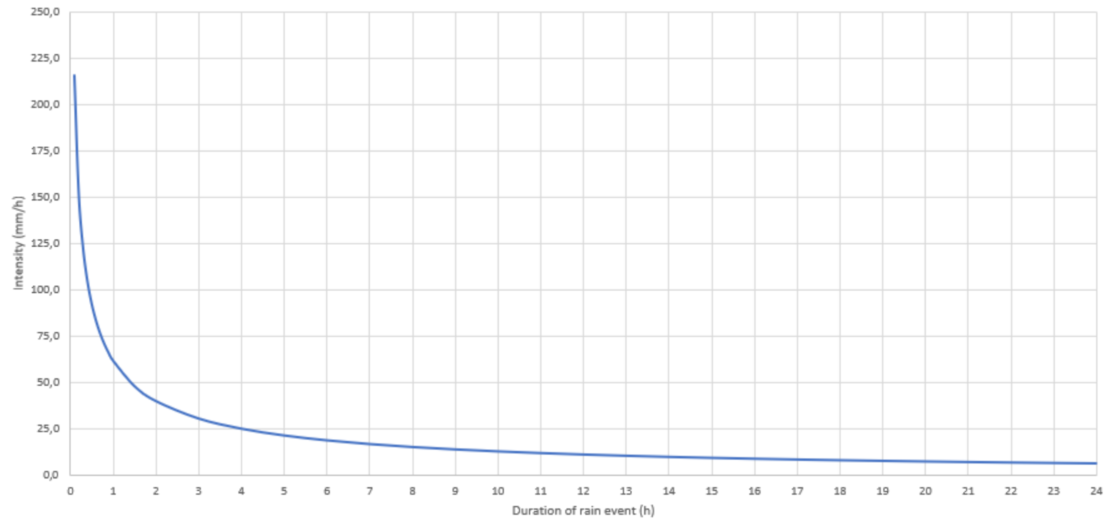

Figura 24. Curva IDF para el periodo de retorno de 15 años.

Respecto a los chaparrones a partir de registros de precipitación observada, se han determinado tres diferentes por el método de la tormenta Gamma de dos parámetros, puesto que el estudio concluye que para cualquier periodo de retorno se asocia tres familias diferentes, todas ellas con magnitudes equivalentes, pero con patrones de intensidad, tiempo y volumen diferentes.

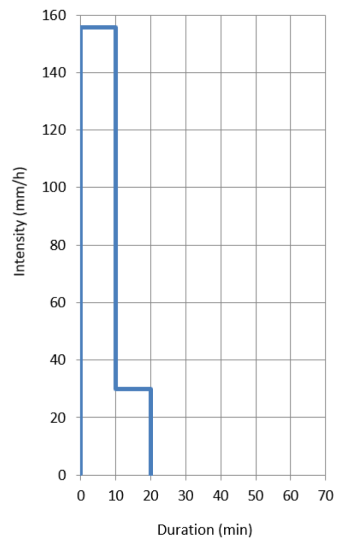

Figura 25. Hietograma chaparrón de diseño, duración corta, Tr = 15 años

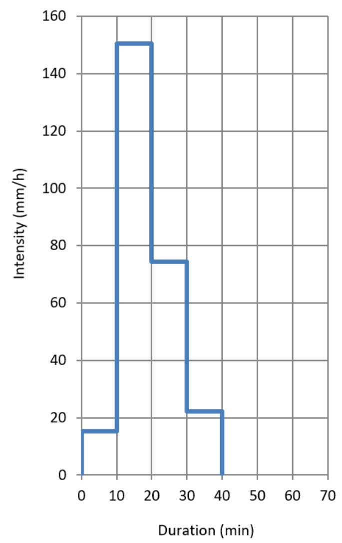

Figura 26. Hietograma chaparrón de diseño duración media, Tr = 15 años

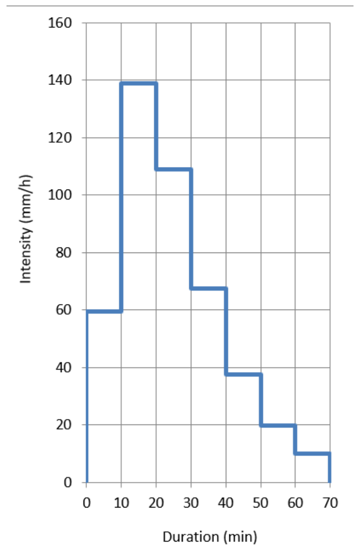

Figura 27. Hietograma chaparrón de diseño duración larga, Tr = 15 años

El gráfico siguiente representa los cuatro chaparrones de diseño a partir de la curva IDF y los determinados por el método G2P

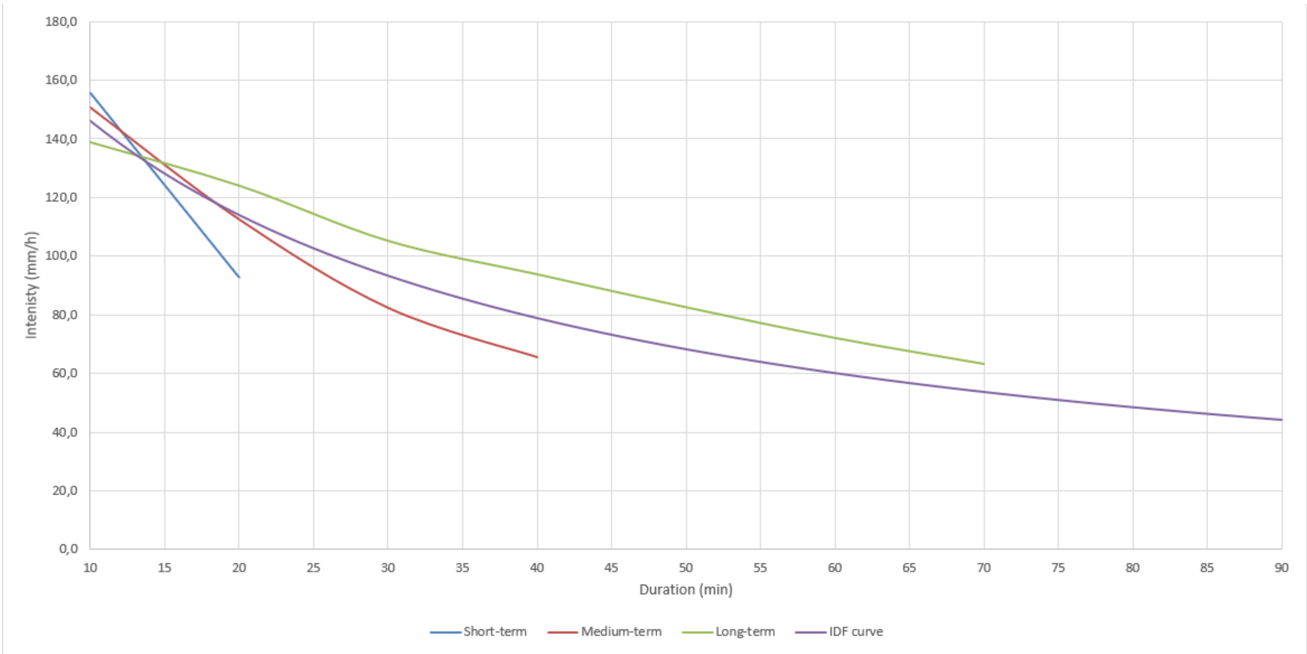

Figura 28. Comparación de chaparrones de diseño para un periodo de retorno de 15 años

7.8 Estado natural

El objeto de los Sistemas de Drenaje Sostenible en este trabajo es, entre otros, el devolver las cuencas del lugar de estudio al estado natural antes de la urbanización de la zona e impermeabilización. Para ello, se deben conocer previamente los parámetros relacionados con el punto de vertido, las cuencas drenantes y el modelo de transformación lluvia-escorrentía utilizado para la simulación del lugar en estado natural.

La zona de estudio se ha dividido en tres cuencas drenantes, la N1, N2 y N3.

Tabla 3. Superficie de las cuencas drenantes

N1	N2	N3	TOTAL
156.712 m2	341.245 m2	696.606 m2	**1.194.563 m2**

En la actualidad, la escorrentía del agua pluvial de las cuencas diverge en el Barranco de la Saleta de manera natural sin sistema de drenaje, conforme el esquema siguiente:

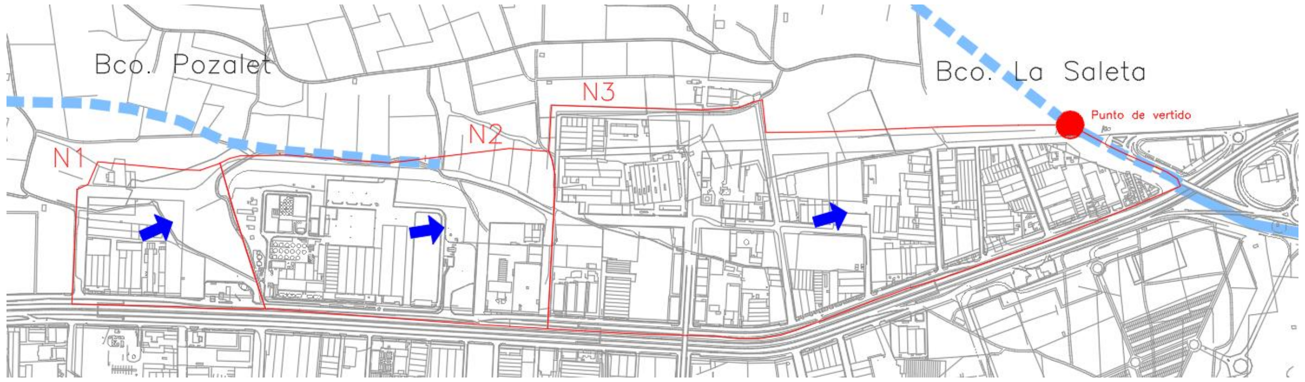

Figura 29. Cuencas drenantes y punto de vertido

La estimación de caudales pico de crecida en estado natural se calcula a partir de los modelos de transformación lluvia-escorrentía. El método utilizado para el modelo de transformación lluvia-escorrentía es aquel recogido en la norma 5.2-IC Drenaje superficial, Orden FOM/298/2016, (Ministerio de Fomento, 2016), propuesto en la revista Ingeniería Civil para la Dirección General de Carreteras de España denominado Método de Témez Modificado (Témez, 1992), cuya expresión es la siguiente:

$$Q_T = \frac{I(T, t_c) * C * A * K_t}{3{,}6}$$

Donde:

- Q_T (m3/s) es el caudal máximo anual correspondiente al período de retorno T.
- I (T, t_c) (mm/h) es la intensidad de precipitación correspondiente al período de retorno considerado T, para una duración del aguacero igual al tiempo de concentración t_c de la cuenca.

- C (adimensional) es el coeficiente medio de escorrentía de la cuenca.
- A (km2) es el área de la cuenca.
- Kt (adimensional) es el coeficiente de uniformidad en la distribución temporal de la precipitación.

Aplicando el método de Témez modificado, se consigue determinar los caudales pico para cada cuenca individualmente y para la totalidad de la zona de estudio

Tabla 4. Caudales máximos en estado natural

	N1	N2	N3	TOTAL
Q (m3/s)	1,77	3,25	4,40	**6,88**

8 PROPUESTA DE SOLUCIONES

Las alternativas propuestas para solucionar la problemática del estudio y que permitirá funcionar como input para la posterior verificación mediante modelización son las siguientes:

- La primera alternativa consta de implementar toda la red de drenaje de forma convencional con un sistema de colectores.

- La segunda alternativa busca mejorar la respuesta del sistema del caso anterior proponiendo diseñar un tanque de tormenta que permita retener y laminar el agua durante los periodos de lluvia para luego introducirla a la red de colectores de aguas residuales de forma controlada.

- La tercera alternativa, basada sobre el sistema de colectores de la primera alternativa, añade sistemas de drenaje sostenibles tales como pavimentos permeables, jardines de lluvia y áreas de biorretención para mejorar la respuesta hidrológica e hidráulica de las cuencas.

- La cuarta y última alternativa, que se diseñará en el caso de que la tercera alternativa no consiga llegar al caudal de vertido umbral propuesto, mejorará la respuesta del sistema de la tercera alternativa añadiendo un tanque de tormenta.

Todo lo mencionado en esta sección se puede encontrar explicado de manera detallada y con mayor profundidad en el *Annex Nº3: Solutions proposal.*

8.1 Criterios y parámetros de diseño

Los criterios de diseño determinantes que se han utilizado para el dimensionamiento hidráulico son:

- El sistema de circulación será preferentemente por **gravedad**, evitando en lo posible los sistemas de elevación e impulsión continua
- El funcionamiento del colector será en **régimen uniforme estacionario**
- El régimen del colector será régimen lento, limitando el número de Froude a $\boldsymbol{Fr < 1}$. Como, en todo caso, el régimen lento estable está asegurado cuando $Fr < 0{,}8$ se intentará limitar a este valor el número de Froude.
- El colector trabajará a **80% del calado** de la sección llena, puesto que ésta es la capacidad óptima de funcionamiento
- La velocidad máxima estará limitada a $\boldsymbol{Vmax = 4\ m/s}$ para evitar daños por fricción
- La velocidad mínima estará limitada a $\boldsymbol{Vmin = 1{,}2\ m/s}$ para evitar la sedimentación de los sólidos arrastrados en suspensión y las obstrucciones
- El diámetro interior mínimo para evitar obstrucciones y facilitar labores de limpieza es $\boldsymbol{Dmin = 335\ mm}$
- La ecuación de pérdida de energía por rozamiento es la dada por la **fórmula de Manning**
- El número de Manning del PVC y del hormigón se considera $\boldsymbol{n = 0{,}011}$, valor conservativo para tener en cuenta la degradación que sufre el colector.
- Se procurará dar **continuidad a la línea de clave** y línea de energía. Por lo que el enrase de colectores se hará obligatoriamente por la clave
- La **línea de energía** se situará siempre por **debajo de la cota del terreno** y se evitaran los remansos con una energía aguas abajo inferior que la energía aguas arriba.
- En el caso de existencia de **resaltos hidráulicos**, se producirá aguas abajo a una distancia $\boldsymbol{d < 10\emptyset}$

Las tuberías que compondrán los colectores serán de dos tipos dependiendo su diámetro, para aquellos diámetros inferiores a 1200 mm se instalarán tuberías de PVC corrugado. Las tuberías con diámetros superiores a 1200 mm serán de PVC rígido, conformadas helicoidalmente con un perfil que presenta una pared estructurada mediante rigidizadores en forma de “T” y refuerzo con perfil de acero galvanizado. Los diámetros escogidos se presentan a continuación:

Tabla 5. Diámetros de la red de colectores

DN (mm)	∅ Externo (mm)	∅ Interno (mm)
400	400	364
500	500	452
630	630	590
800	800	775
1000	1000	970
1200	1200	1103
1300	1300	1268
1400	1400	1368
1500	1500	1468
1600	1600	1568
1700	1700	1668
1800	1800	1768
1900	1900	1868
2000	2000	1968
2100	2100	2068
2200	2200	2168
2300	2300	2268
2400	2400	2368
2500	2500	2468
2600	2600	2568
2700	2700	2668
2800	2800	2768
2900	2900	2868
3000	3000	2968

El diseño de la red de sistemas de drenaje sostenible se determinará a partir del control volumétrico para conseguir llegar a los objetivos de cantidad y calidad. El criterio de cantidad a tratar para conseguir la laminación de los caudales generados por episodios de lluvias viene dado por el valor V_{80}, mientras que el criterio de calidad corresponde al de poder almacenar un volumen de escorrentía correspondiente al valor V_{90}. En adición, se pretenderá mantener el patrón de drenaje natural del ámbito, intentado que los caudales de evacuación al punto de vertido no sean superiores a los caudales calculados en estado natural para un periodo de retorno de 15 años. Este objetivo es mucho más ambicioso y, por lo tanto, tendrá como volúmenes de almacenamiento para gestionar mayores. Determinando los siguientes valores para las cuencas de estudio:

Tabla 6. Valor V80, V90 y máximo del almacenamiento necesario para gestionar un evento de T=15 años

	N1	N2	N3
Volumen almacenamiento V80 (m3)	1.988	4.609	9.927
Volumen almacenamiento V90 (m3)	3314	7682	16544
Volumen almacenamiento chaparrón largo (m3)	9.756	22.617	48.707

Además, se ha realizado una comprobación del control de calidad mediante el Método de los índices de niveles de riesgo de contaminación, indicando que la mitigación de contaminantes por parte de los SUDS debe ser suficiente para contrarrestar el índice de riesgo de contaminación del área industrial.

Tabla 7. Niveles de contaminación en áreas industriales. Fuente: Woods Ballard et al., 2015

Sólidos suspendidos totales	Metales	Hidrocarburos
0,8	0,8	0,9

Común a todos los escenarios, se presenta la división en subcuencas de la zona de estudio en subcuencas privadas, subcuencas de calle y subcuencas de zonas verdes.

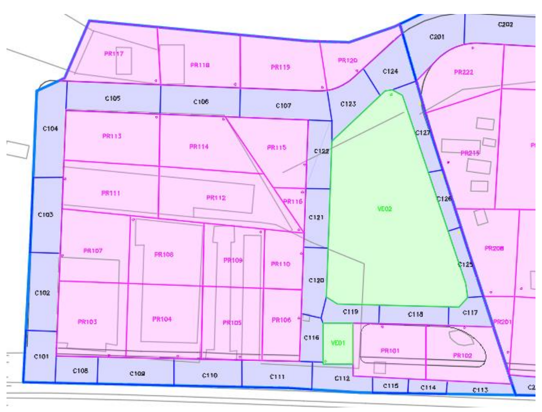

Figura 30. División de subcuencas en la cuenca N1

Figura 31. División de subcuencas en la cuenca N2

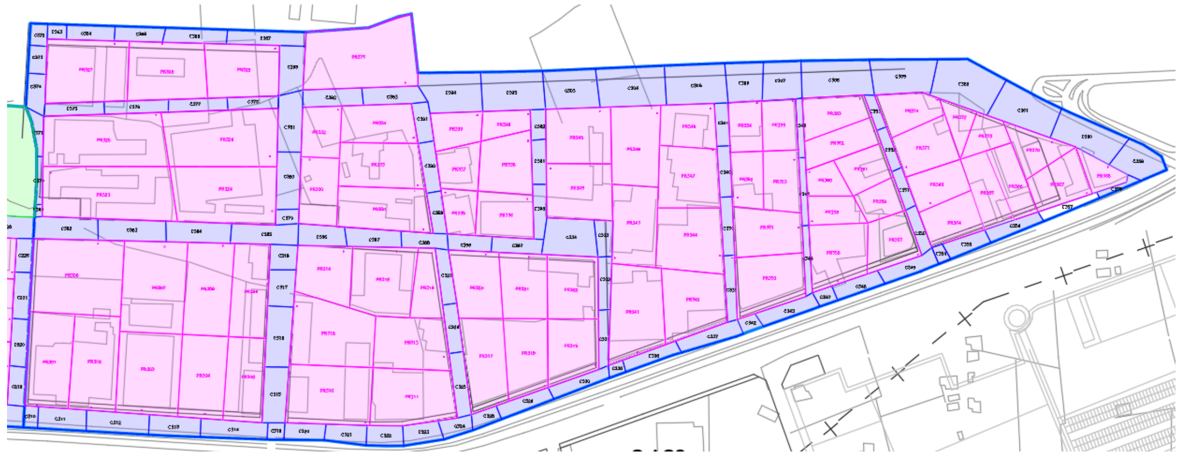

Figura 32. División de subcuencas en la cuenca N3

A partir de esta división de subcuencas, se ha determinado un coeficiente de escorrentía a cada tipo, indicados a continuación:

Tabla 8. Coeficientes de escorrentía para la zona de estudio

Superficie	Coeficiente de escorrentía
Calle	0,95
Subcuenca privada	0,75
Zona verde	0,30
Pavimento permeable	0,70
Jardín de lluvia	0,30
Balsa de infiltración	0,30

8.2 Alternativa 1. Solución convencional

La primera alternativa consta de implementar toda la red de drenaje de forma convencional con un sistema de colectores. No puede considerarse la alternativa 0, la cual corresponde a no actuar, porque esta alternativa supone una mejora sustancial de la situación actual, ya que hoy en día no existe sistema de drenaje en la zona de estudio y esta alternativa mejoraría el control y conducción de la escorrentía generada.

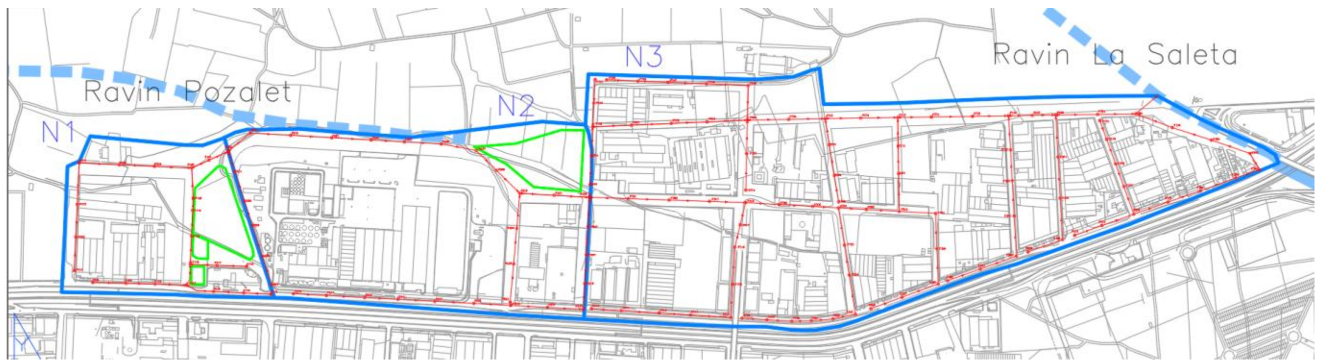

Figura 33. Propuesta de la alternativa 1

8.3 Alternativa 2. Tanque de tormenta

La alternativa 2 se compone de la red convencional de drenaje anterior, la cual finaliza en un tanque de tormenta que permite laminar y almacenar el agua de escorrentía para disminuir el caudal pico y controlar el régimen de sueltas hacia el Barranco de La Saleta.

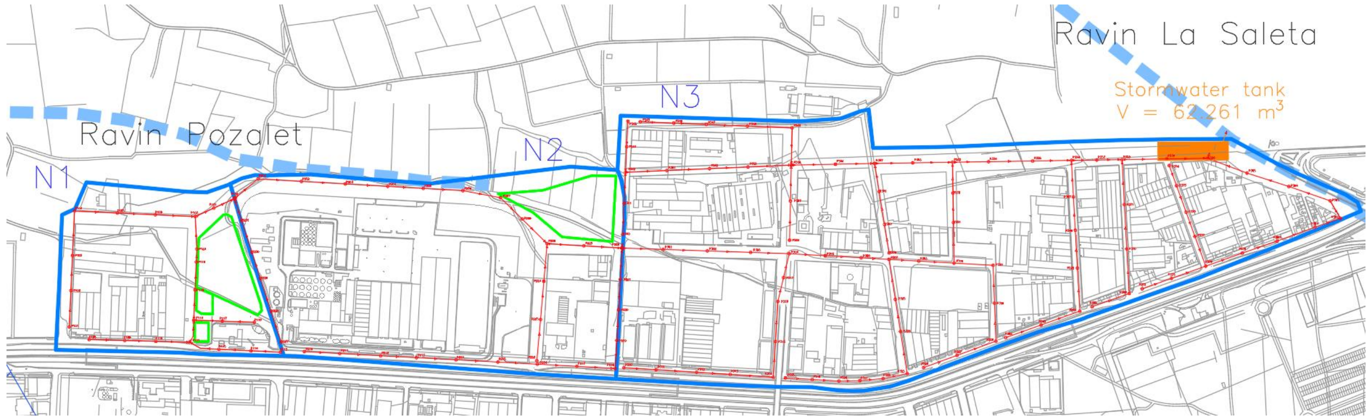

Figura 34. Propuesta de la alternativa 2

El tanque de tormenta esta dimensionado para el volumen que se quiere almacenar y el caudal pico objetivo que se quiere verter al medio receptor. El sistema de vaciado del tanque se realizará mediante un sistema de bombeo y las dimensiones de este tanque serán las siguientes:

$$S_{tanque} = \mathbf{6.220\ m2}$$
$$h_{tanque} = \mathbf{10,0\ m}$$

$$V_{tanque} = \mathbf{62.261\ m}$$

8.4 Alternativa 3. SuDS

La alternativa 3 busca implementar una solución que incorpore sistemas de drenaje sostenible, analizando los condicionantes de la zona de estudio se ha determinado que los SuDS que mejor se incorporan al polígono industrial son los pavimentos permeables, los jardines de lluvia y las balsas de infiltración.

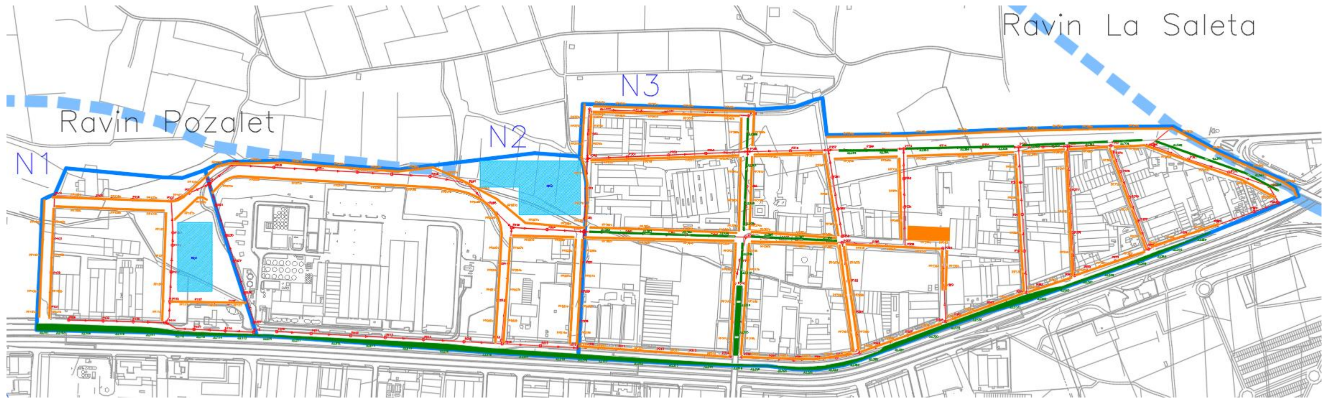

Figura 35. Propuesta de la alternativa 3

Mediante la cadena de SuDS se consigue llegar a índices de mitigación de contaminantes total superiores a los niveles de riesgo de contaminación para áreas industriales

Tabla 9. Indicadores de mitigación total en la zona de estudio

Sólidos Suspendidos Totales	Metales	Hidrocarburos
1	0,85	1

Tabla 10. Niveles de riesgo de contaminación para áreas industriales. Fuente: Woods Ballard et al., 2015

Sólidos Suspendidos Totales	Metales	Hidrocarburos
0,8	0,8	0,9

8.5 Alternativa 4. SuDS con tanque de tormenta

La alternativa 4 surge a partir de los resultados de la alternativa 3. El desarrollo de esta alternativa se realiza en el caso de que la tercera alternativa no consiga llegar al caudal de vertido en estado natural, ofreciendo un escenario que tenga una mejor respuesta hidráulica a la problemática.

Esta alternativa se desarrolla partiendo de los sistemas de drenaje sostenible de la alternativa 3 y el dimensionamiento del tanque de tormenta de la alternativa 2 para conseguir llegar al umbral de caudal de vertido objetivo del trabajo.

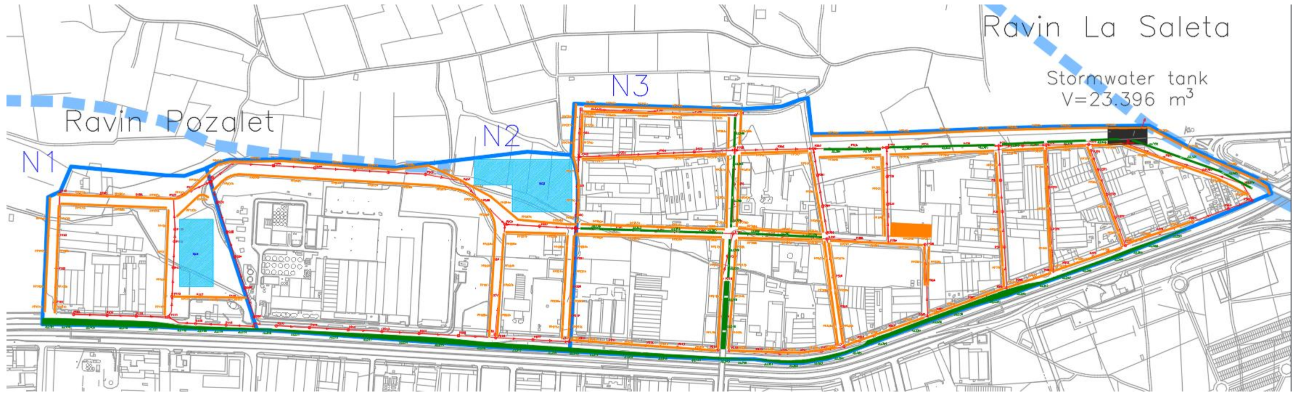

Figura 36. Propuesta de la alternativa 4

El tanque de tormenta en este caso es de dimensiones muy inferiores a la alternativa 2.

$$S_{tanque} = \mathbf{3.655\ m2}$$
$$h_{tanque} = \mathbf{6,4\ m}$$

$$V_{tanque} = \mathbf{23.396\ m}$$

El sistema de vaciado se realiza mediante un bomba tipo CPH 350-360 con un caudal de vaciado de 600 l/s para vaciar el tanque de tormenta en menos de medio día.

9 MODELIZACIÓN DE LAS PROPUESTAS

Los modelos matemáticos de un sistema físico son una simplificación de la realidad que conserva las características principales. Están compuestos por entradas "inputs", parámetros, ecuaciones y variables de estado.

La modelización matemática del drenaje urbano tiende a representar los procesos de producción de escorrentía por precipitación, transporte de escorrentía en superficie y propagación de la escorrentía en la red de colectores. En el caso de nuestro estudio, el uso de un modelo matemático es principalmente para el diseño de una nueva red a partir de cálculos de dimensionamiento realizados, y en su caso, optimizar este diseño.

La modelización tiene como bases teóricas el dimensionamiento hidráulico influenciado por:

- **Las ecuaciones de Saint-Venant:** son las ecuaciones que gobiernan el flujo transitorio unidimensional en lámina libre.
- **La ecuación de continuidad:** que establece el balance de masas en un determinado elemento de control
- **La ecuación de cantidad de movimiento:** establece el equilibrio dinámico del elemento de control

El programa utilizado en este estudio es el software SWMM, el cual utiliza el **método de diferencias finitas explícito**. El método de diferencias finitas es el más utilizado entre los programas de modelización de drenaje urbano y consiste en discretizar el espacio bidimensional (x, t) en una malla definida a partir de un Δx y un Δt y aproximar las derivadas parciales de las ecuaciones.

Como el apartado anterior, un análisis más en profundidad sobre el programa y la modelización realizada aparece en el *Annex Nº3: Solutions proposal.*

9.1 Modelo de infiltración

El modelo de infiltración utilizado para el estudio es el modelo empírico del SCS (Soil Conservation Service) del **número de curva (CN)**. Además del modelo CN, el programa SWMM ofrece cuatro modelos más. El modelo Horton, Horton modificado, Green y Ampt y Green y Ampt modificado. Aunque estos cuatro últimos modelos reproducen con mucha más precisión los procesos involucrados en la producción de escorrentía, tienen el inconveniente de peso para su uso en modelos de drenaje urbano de que dependen de demasiados parámetros, en ocasiones, difíciles de estimar. Debido a ello, se ha optado por elegir el modelo CN para la modelización de la infiltración.

El CN varía entre 0 y 100, siendo este último valor el que corresponde a una escorrentía del 100%. La relación entre el CN y los parámetros S y P_0 es la siguiente:

$$S = \frac{25400}{CN} - 254$$

$$P_0 = \frac{5080}{CN} - 50{,}8$$

Donde:

- S (adimensional) es la diferencia máxima potencial entre la lluvia caída y la escorrentía generada
- P0 (mm) es el umbral a partir del cual se genera escorrentía.

Los valores tomados para el CN en el estudio son los siguientes:

Tabla 11. Número de curva por zona. Fuente: Rossman L. A., 2015

Zonificación	CN – Cond. II
Impermeable – Calles y calzada	98
Edificación - Zonas privadas industriales	91
Permeable - Parques y espacios verdes	74

9.2 Modelo hidráulico de transporte

El modelo hidráulico de transporte utilizado para resolver las ecuaciones es el modelo de **onda dinámica**, puesto que resuelve ecuaciones completas unidimensionales de Saint-Venant y, por lo tanto, genera resultados más precisos. El inconveniente de este modelo es que exige incrementos de tiempos menores, del orden de 1 minuto, haciendo que la simulación tarde más en ejecutarse. Sin embargo, la simulación de la estudio no posee tantos elementos, condiciones ni parámetros como para que esto sea suponga una limitación.

9.3 Elementos del modelo

La red de drenaje se ha definido en el programa SWMM mediante los elementos geométricos que ofrece el programa. En la modelización del caso de estudio se utilizarán de entre todas las opciones del programa los subcuencas, uniones, puntos de vertido, depósitos, conducciones, pluviómetros y controles LID.

- Las **subcuencas** son unidades hidrológicas del terreno cuya topografía y elementos del sistema de drenaje conducen la escorrentía directamente hacia un punto de descarga.

- Las **uniones** son nudos del sistema de drenaje donde se conectan las diferentes líneas entre sí.

- Los **puntos de vertido** son nudos terminales del sistema de drenaje utilizados para definir las condiciones de contorno finales aguas abajo del sistema en el caso de utilizar el modelo de flujo de la onda dinámica

- Los **depósitos** son nudos del sistema de drenaje con la capacidad para almacenar determinados volúmenes de agua.

- Las **conducciones** son tuberías o canales por los que se desplaza el agua desde un nudo a otro del sistema de transporte

- Los **pluviómetros** suministran los datos de entrada de las precipitaciones que ocurren sobre una o varias de las cuencas definidas en el área de estudio.

La modelización de los sistemas de drenaje sostenible, llamados **LID** (Low Impact Development) en el programa, se realiza siguiendo el siguiente esquema conceptual donde, dependiendo de la tipología de SUDS, el elemento estará compuesto de capas y flujos diferentes.

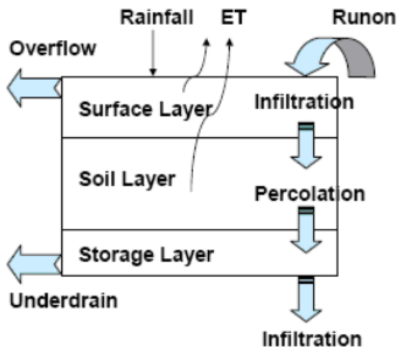

Figura 37. Diagrama conceptual de la representación de los LID. Fuente: Rossman L. A., 2015

9.4 Modelización alternativas

Se ha procedido a implementar las diferentes alternativas dentro del programa SWMM a partir de los elementos y los parámetros de diseño determinados previamente. La representación de la modelización en el programa SWMM se reconoce de la siguiente forma:

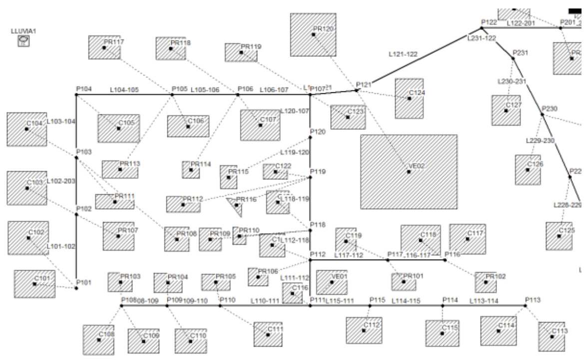

Figura 38. Modelización cuenca N1. Alternativa 1

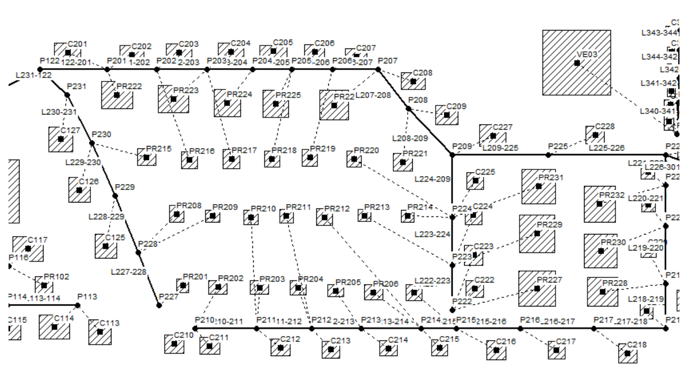

Figura 39. Modelización cuenca N2. Alternativa 1

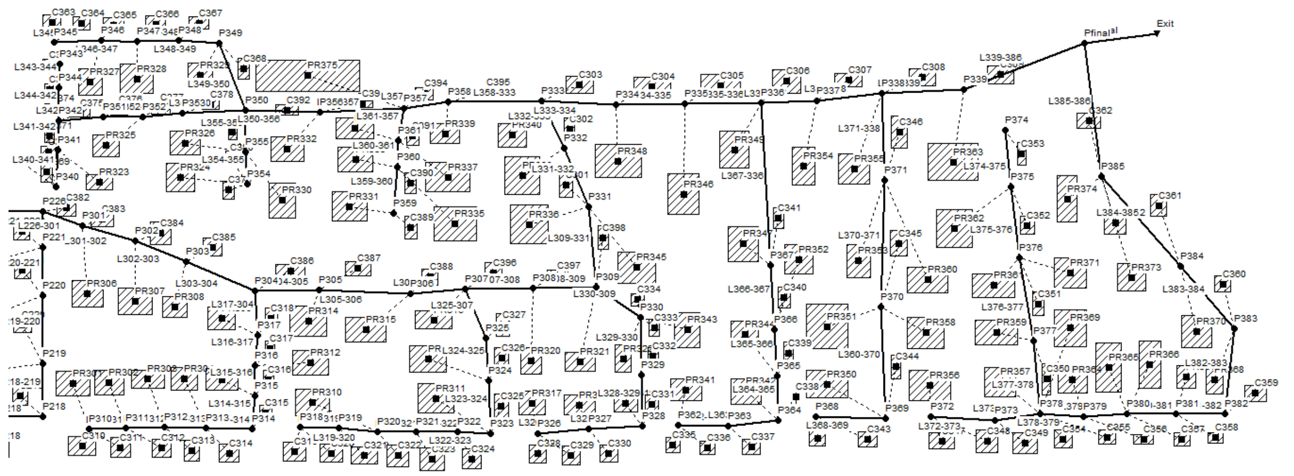

Figura 40. Modelización cuenca N3. Alternativa 1

La representación de la alternativa 2 es muy similar a la representación de la modelización 1, pero cambiando el nodo de conexión final por un depósito.

La representación de la modelización del escenario 3 guarda los conductos de la red de drenaje de la modelización 1, pero se han modificado las cuencas para añadir los SUDS en las parcelas. De este modo, se han reducido las áreas de los subcatchment que representan a las parcelas y se ha añadido otro subcatchment que represente el SuDS. Las cuencas N1 y N2 tienen un sistema de lógica de conexiones semejante, mientras que la cuenca N3 es un poco más compleja.

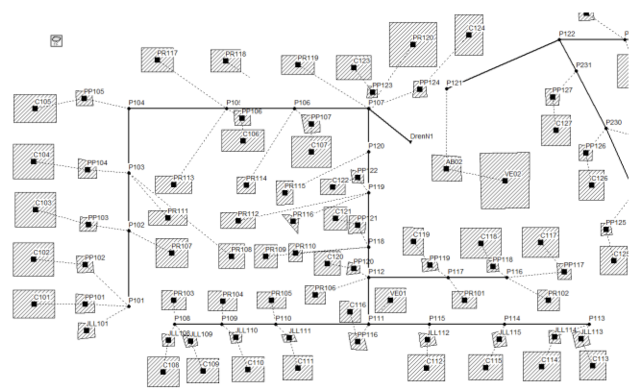

Figura 41. Modelización cuenca N1. Alternativa 3

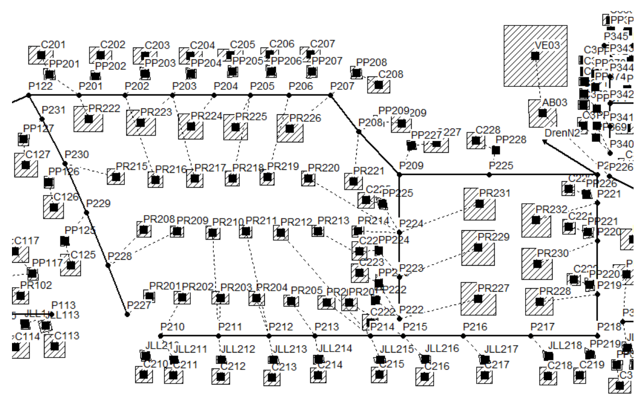

Figura 42. Modelización cuenca N2. Alternativa 3

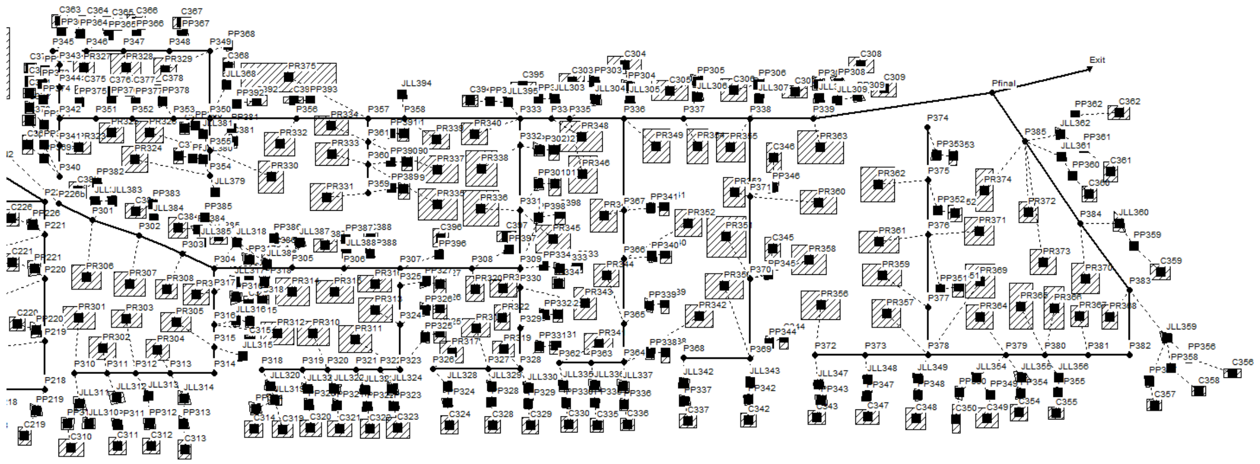

Figura 43. Modelización cuenca N3. Alternativa 3

La representación de la alternativa 4 se hace a partir de la modelización 3 y se añade en el tramo final un depósitos con el volumen suficiente para poder gestionar el agua de escorrentía de tal forma que el vertido final sea igual al del estado natural.

Se han recopilado los resultados más remarcables de todas las alternativas, correspondientes al caudal pico y volumen vertido, para poder hacer una comparación conjunta.

Tabla 12. Comparación caudales vertidos alternativas

Chaparrón de diseño	Caudal pico (m3/s)			
	Alt 1	Alt 2	Alt 3	Alt 4
Corto	24,49	0,00	12,69	0,00
Medio	29,71	0,00	14,14	0,00
Largo	33,29	6,88	15,91	6,88

Tabla 13. Reducción caudales vertidos alternativas

Chaparrón de diseño	Reducción caudal pico			
	Alt 1	Alt 2	Alt 3	Alt 4
Corto	-	100%	48,2%	100%
Medio	-	100%	52,4%	100%
Largo	-	79,3%	52,2%	79,3%

Tabla 14. Comparación volúmenes vertidos alternativas

Chaparrón de diseño	Volumen vertido (m3)			
	Alt 1	Alt 2	Alt 3	Alt 4
Corto	25.945	0	13.101	0
Medio	40.208	0	20.667	0
Largo	74.809	18.222	39.584	17.868

10 ANÁLISIS MULTICRITERIO COMPARATIVO

El análisis comparativo de las proposiciones para poder encontrar la solución óptima se ha realizado mediante el software E2Stormed, el cual funciona como una DST (Decision Support Tool) que ayuda en el proceso de decisión analizando ventajas y desventajas de sistemas de drenaje para la gestión de agua pluvial teniendo en cuenta criterios financieros, hidráulicos, energéticos, medioambientales y sociales.

El análisis multicriterio de esta sección se puede encontrar explicado de manera detallada y con mayor profundidad en interpretación de resultados en el *Annex Nº4: Comparative multi-criterial analysis.*

Para la realización del análisis se introduce en primer lugar los datos para cada propuesta respecto a los siguientes aspectos:

- Suministro de agua
- Volumen de escorrentía
- Bombeo y tratamiento
- Calidad del agua
- Protección frente a inundaciones
- Aislamiento de edificios
- Servicios de ecosistemas

Posteriormente, se incluyen todas las infraestructuras de cada alternativa, incluyendo el coste, la energía consumida y la emisión de contaminantes para el proceso de construcción y mantenimiento.

De este modo, se obtiene como resultado para cada alternativa:

Tabla 15. Resultados de coste, energía consumida y emisiones emitidas para cada alternativa

Infraestructura	CONSTRUCCIÓN			MANTENIMIENTO		
	Coste (€)	Energía consumida (kWh)	Emisiones (kgCO2e)	Coste (€/año)	Energía consumida (kWh/año)	Emisiones (kgCO2e)
Alternativa 1	20.203.700	57.042.850	17.962.588	295.348	6.055	1.594
Alternativa 2	51.334.200	109.921.495	34.712.042	388.740	6.063	1.596
Alternativa 3	22.252.072	53.657.674	16.895.614	550.329	5.380	1.417
Alternativa 4	33.950.072	73.527.662	23.189.606	585.423	5.388	1.419

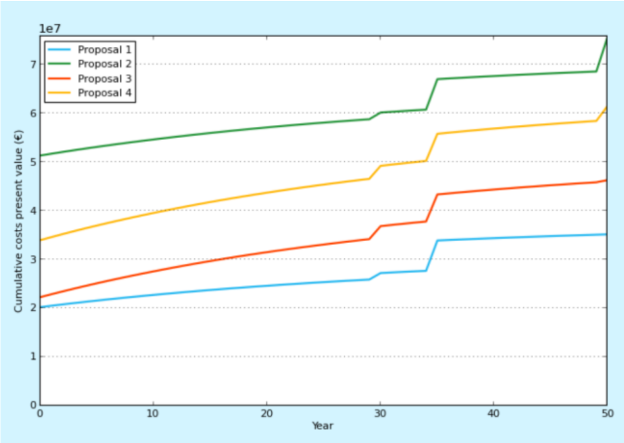

Figura 44. Costes totales acumulados en valor presente durante la vida útil

Tras el análisis de los resultados se determina que la solución más económica es la propuesta 1, como cabía esperar, ya que el resto de las proposiciones son una modificación de la propuesta 1 y por lo tanto su coste de construcción y mantenimiento iba a ser mayor. Teniendo en cuenta que los costes de construcción y mantenimiento son de una magnitud mucho mayor que el resto de los costes, la diferencia de coste de construcción y mantenimiento entre alternativas juega un papel crucial en la economía de las soluciones.

E2stormed también ofrece los resultados comparados del consumo de energía a lo largo del proyecto. Los resultados identifican las propuestas 2 y 4 como aquellas que más energía consumen. Esto es debido principalmente a la implantación de un sistema de bombeo para elevar el agua del tanque de tormenta a la cota donde se encuentra la red de aguas residuales. La propuesta 2 consume más energía porque el tanque es de mayores dimensiones y debe extraer más agua a un profundidad mayor.

La comparativa de las emisiones acumuladas de cada proposición se presenta en la siguiente gráfica. Las emisiones de las propuestas se presentan en concordancia con las de consumo energético, ya que el consumo de energía es uno de los mayores emisores en CO2 equivalente. En los escenarios propuestos, en ningún caso las emisiones disminuyen con el paso del tiempo.

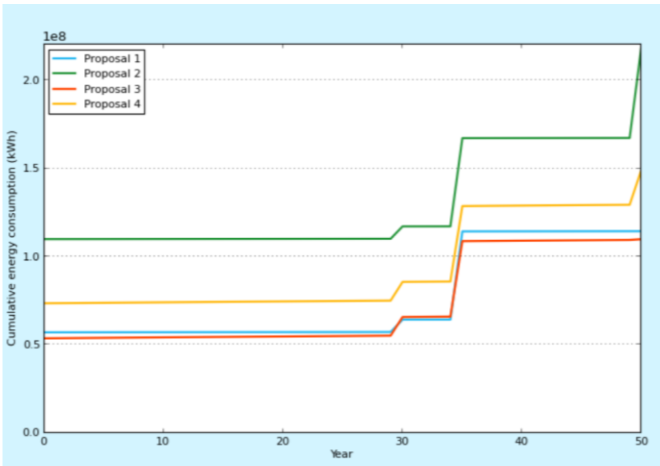

Figura 45. Energía consumida acumulada durante vida útil

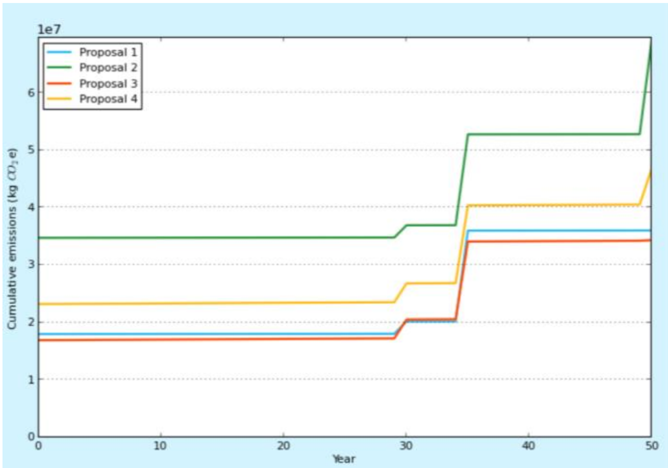

Figura 46. Emisiones durante la vida útil

A continuación, se deben elegir los criterios que vayan a determinar la solución óptima y añadirle un peso a cada uno de ellos, en nuestro caso se han considerado seis como los determinantes a la hora de elección de alternativas

Tabla 16. Pesos elegidos para cada criterio

Criterio		Peso
Coste neto de la gestión del agua de lluvia	C1	40%
Caudal pico de vertido	C2	35%
Calidad global del agua vertida	C3	7%
Evaluación de los ecosistemas de servicio	C4	7%
Energía consumida neta en la gestión de agua de lluvia	C5	5,5%
Emisiones en la gestión de agua de lluvia	C6	5,5%

De esta forma se obtiene para cada alternativa los resultados en forma de diagrama correspondientes a cada criterio:

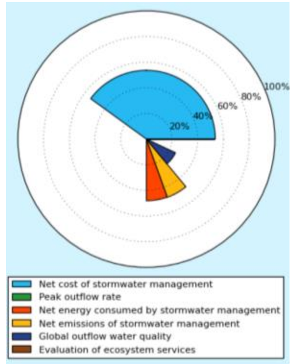

Figura 47. Diagrama resultado del análisis multicriterio. Alternativa 1

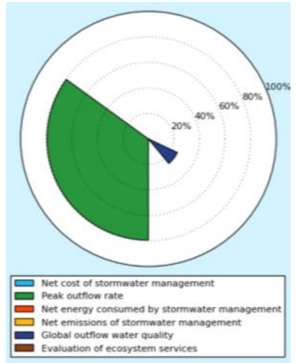

Figura 48. Diagrama resultado del análisis multicriterio. Alternativa 2

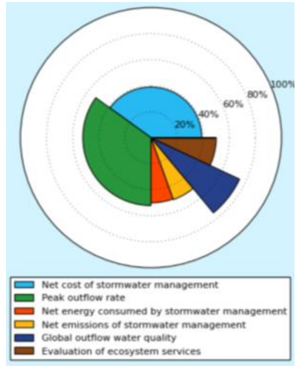

Figura 49. Diagrama resultado del análisis multicriterio. Alternativa 3

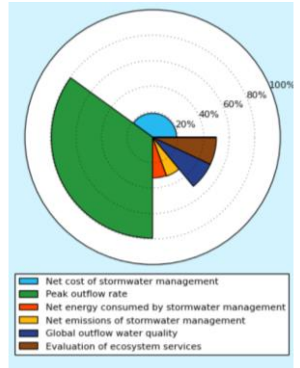

Figura 50. Diagrama resultado del análisis multicriterio. Alternativa 4

Finalmente, a partir de estos resultados obtenemos la global score de cada alternativa cuando se multiplica los resultados anteriores por el peso de cada criterio. Esto nos permite comparar de todos los escenarios.

Tabla 17. Puntuación global del análisis multicriterio

Criterio	Alternativa 1	Alternativa 2	Alternativa 3	Alternativa 4
Coste neto de la gestión del agua de lluvia (%)	21,44	0,00	15,55	7,49
Caudal pico de vertido (%)	0,00	27,77	18,27	27,77
Calidad global del agua vertida (%)	1,75	1,75	5,25	3,5
Evaluación de los ecosistemas de servicio (%)	0,00	0,00	3,5	3,5
Energía consumida neta en la gestión de agua de lluvia (%)	2,64	0,00	2,76	1,76
Emisiones en la gestión de agua de lluvia (%)	2,65	0,00	2,78	1,79
TOTAL (%)	**28,49**	**29,52**	**48,12**	**45,81**

Su representación gráfica se realiza mediante un histograma que identifica los resultados de cada criterio aplicado a cada alternativa, se presenta en la siguiente imagen:

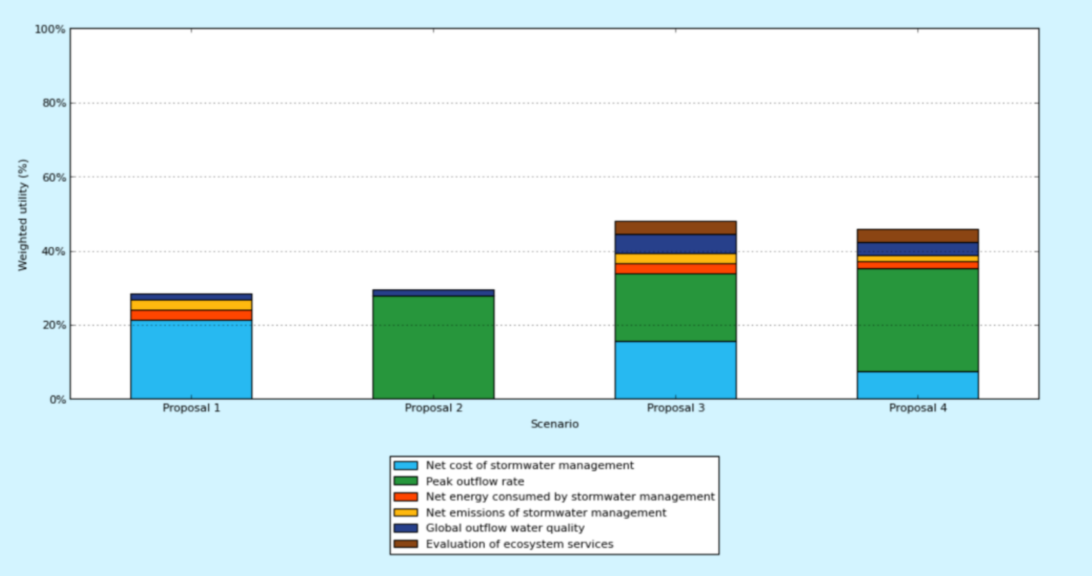

Figura 51. Histograma de los resultados globales del análisis multicriterio

Tras el análisis multicriterio se llega a la conclusión de que **la solución óptima es la alternativa 3,** donde se implementan los sistemas de drenaje sostenible pero no incluye un tanque de tormenta por lo que no se llega al caudal pico objetivo en estado natural. No obstante, la alternativa 4 se localiza muy próxima a ella. Analizando en detalle los motivos por los que la alternativa 3 se localiza en una posición superior que la alternativa 4, se ha concluido que la alternativa 3 se coloca en primera posición gracias a los beneficios de su sistema de drenaje en cuanto a energía, emisiones y medioambiente, aunque en los aspectos conjuntos de hidráulica y finanzas sea inferior a la alternativa 4.

Tabla 18. Análisis detallado entre alternativa 3 y 4

Criterio	Alternativa 3	Alternativa 4
Criterios financieros e hidráulicos (%) C1 + C2	33.82	35.26
Criterios medioambientales y energéticos (%) C3 + C4 + C5 + C6	14.3	10.55
TOTAL (%)	**48,12**	**45,81**

Esto indica que, para nuestro análisis, el beneficio que supondría la construcción de un tanque de tormenta en cuanto a caudal vertido hace que sea la mejor opción incluso teniendo en cuenta sus costes. Sin embargo, la implantación añadida del tanque y su sistema de bombeo, con sus consecuencias en cuanto a criterios energéticos y medioambientales, hacen que esta alternativa caiga en segunda posición. Por último, se interpreta que la proximidad de las propuestas 3 y 4 da a entender que el orden de prioridad de las alternativas es considerablemente sensible a los pesos que se han dado a los criterios.

11 DESARROLLO DE LA SOLUCIÓN ADOPTADA

Identificando la alternativa 3 como la óptima, se ha desarrollado la solución adoptada, la cual se puede encontrar con detalle en el *Annex Nº5: Development of the optimal solution*. A continuación, se presentan las características más remarcables de las secciones de los sistemas de drenaje sostenible a implementar en la zona de estudio.

11.1 Pavimentos permeables

La sección de los pavimentos permeables estará compuesta por las siguientes capas:

- Capa superficial de hormigón poroso
- Capa subsuperficial de gravillín
- Lámina geotextil
- Subbase de grava
- Geomembrana impermeable
- Tubo dren

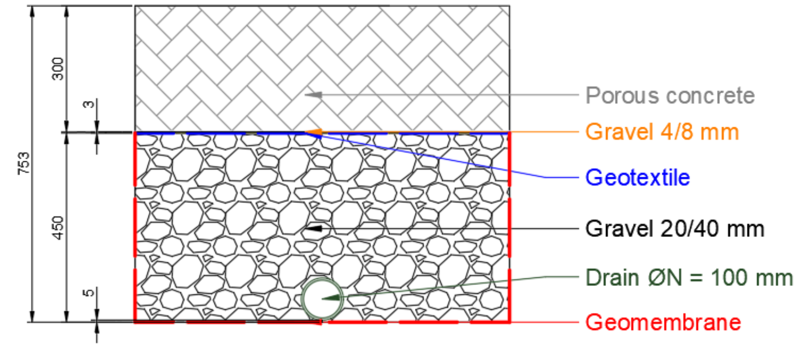

Figura 52. Sección tipo de los pavimentos permeables

La capa de superficie de hormigón poroso será la primera capa superior de la sección que estará en contacto con los vehículos. Esta capa tendrá las siguientes características:

- El espesor de la capa será de 300 mm.
- El índice de huecos para la mezcla porosa corresponderá al 25%.
- Su rigidez estará entre 25 y 45 GPa, siendo su valor común de 38 GPa.
- El peso específico del hormigón poroso estará entre 1700 y 2000 kg/m3.
- Se buscará una granulometría sin finos o, por menos, con un contenido bajo. Identificando un tamaño mínimo de árido de 5mm. El tamaño de árido máximo será de 20 mm.
- El conglomerante utilizado será el cemento hidráulico, con un contenido de 267-326 kg/m3 y un ratio agua cemento de 0,26-0,38. El cemento será modificado con aditivos de humo de sílice, superplastificante y polímeros orgánicos para pasar de resistencias de compresión medias de 20 MPa a asegurar una resistencia a compresión superior a 45 MPa.
- La resistencia a flexo-tracción mínima será de 2,5 MPa.
- La permeabilidad del hormigón poroso estará entorno los 250 mm/h

Se colocará una subcapa de gravillín debajo de la capa superficial de hormigón poroso con un espesor de 30 mm y un tamaño de árido mínimo de 4mm y máximo de 8mm, incrementando el tamaño del árido con la profundidad.

Entre la capa superficial y la capa de almacenamiento se colocará una lámina de geotextil no tejido, que realizará la función de filtro y separación para evitar el traspaso de finos y la pérdida de soporte en la base. Esta lámina además se encarga de retener la mayor parte de los contaminantes arrastrados por el agua filtrada. Sus características serán:

- Masa por unidad de superficie entre 125 y 160 gr/m2.
- Permeabilidad vertical entre 100 y 130 mm/s.

La capa de almacenamiento estará compuesta por las siguientes especificaciones:

- El espesor será de 450 mm.
- Los áridos serán artificiales y procedentes de cantera
- La granulometría deberá ser lo más uniforme posible, procurando eliminar los finos menores de 2mm.
- El índice de huecos será de 50%.
- El coeficiente de Los Ángeles deberá ser inferior a 30.
- El equivalente de arena será superior a 40.
- La granulometría será entre 20 y 40 mm.

Por último, se colocará una capa de geomembrana para la impermeabilización de toda la estructura y de este modo impedir la infiltración. Esta lámina deberá asegurar un total sellado de la explanada que permita contar una estructura estanca que funcione como depósito de almacenamiento del agua filtrada.

La salida del agua almacenada ser realizará mediante un tubo dren de PVC embebido en las gravas a 5 mm de la base. El diámetro nominal del tubo será de 100mm y estará revestido con un geotextil que evite obstrucciones en sus ranuras.

11.2 Jardines de lluvia

Los jardines de lluvia presentan dos secciones tipo. Una con geomembrana y dren y otra sin ninguno de los dos, permitiendo así la infiltración.

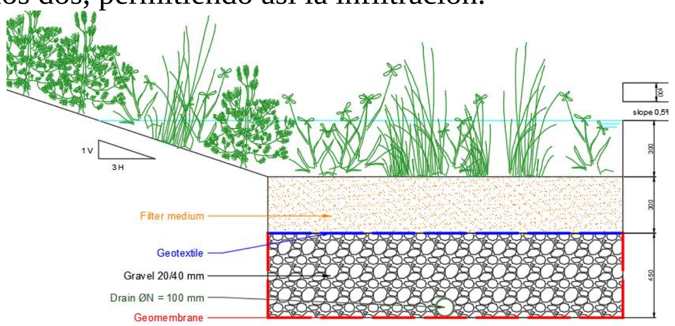

Figura 53. Sección tipo de los jardines pluviales sin infiltración

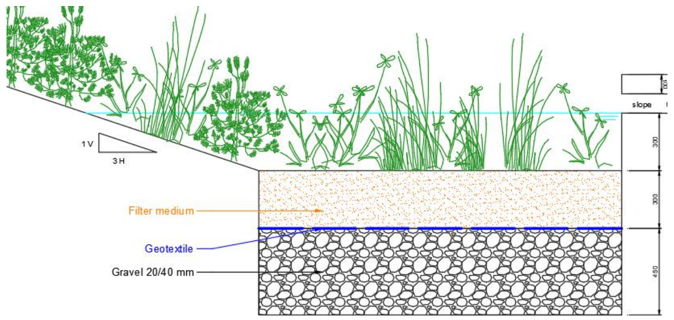

Figura 54. Sección tipo de los jardines pluviales con infiltración

Las secciones tipo estarán compuestas por las siguientes capas:

- Superficie libre
- Medio filtrante
- Lámina geotextil
- Subbase de grava
- Geomembrana impermeable (si no hay infiltración)
- Tubo dren (si no hay infiltración)

La superficie libre disponible en los jardines de lluvia permite almacenar a modo de depósito una altura de agua de hasta 300mm. Para los eventos que excedan al de diseño, se incluye una estructura de alivio, normalmente un aliviadero de tipo buzón, con un altura de 100 mm para dirigir la escorrentía aguas abajo en condiciones de seguridad, con una pendiente del 0,5%. Se incluirá también cierto resguardo entre la cota superior del aliviadero y la cota de acabado de la superficie adyacente de 100 mm. La superficie libre estará cubierta con vegetación autóctona, con una densidad mínima de 6 ud/m2 y una fracción volumétrica mínima de vegetación de 0,1/m3.

El medio filtrante tiene las siguientes características:

- El espesor será de 300 mm.
- La permeabilidad será de 250 mm/h
- La porosidad será del 0,5
- El contenido en materia orgánica será entre 3-5%
- El pH estará entre 5.5 y 8.5
- La conductividad eléctrica será menor que 3300 microS/cm

Entre la medio filtrante y la capa de almacenamiento se colocará una lámina de geotextil no tejido para asegurarse que las partículas del suelo no pasan de una capa a otra. Esta lámina posee las mismas funciones que en los pavimentos permeables y las mismas características:

- Masa por unidad de superficie entre 125 y 160 gr/m2.
- Permeabilidad vertical entre 100 y 130 mm/s.

La subbase de grava debe ser más permeable que el medio filtrante.

- El espesor será de 450 mm
- El tamaño de árido se colocará entre 20 y 40 mm
- El índice de huecos será de 50%.
- Los áridos serán artificiales y procedentes de cantera

En el caso de que el jardín de lluvia no permita la infiltración, es decir cuando están en la cabeza de la cadena de mitigación de contaminantes, se dispondrá una lámina de geomembrana que impermeabilice la estructura.

En el caso de que el jardín de lluvia no permita la infiltración, la salida del agua almacenada ser realizará mediante un tubo dren de PVC embebido en las gravas a 5 mm de la base. El diámetro nominal del tubo será de 100mm y estará revestido con un geotextil que evite

11.3 Balsas de infiltración

Las áreas de infiltración tienen una sección similar a los jardines de lluvia de que permiten la infiltración, excepto que estas áreas tienen una superficie libre que permite un mayor calado del agua y que no tienen función de biorretención, ya que los elementos SUDS de biorretención no pueden tener una extensión mayor que 0,8 ha.

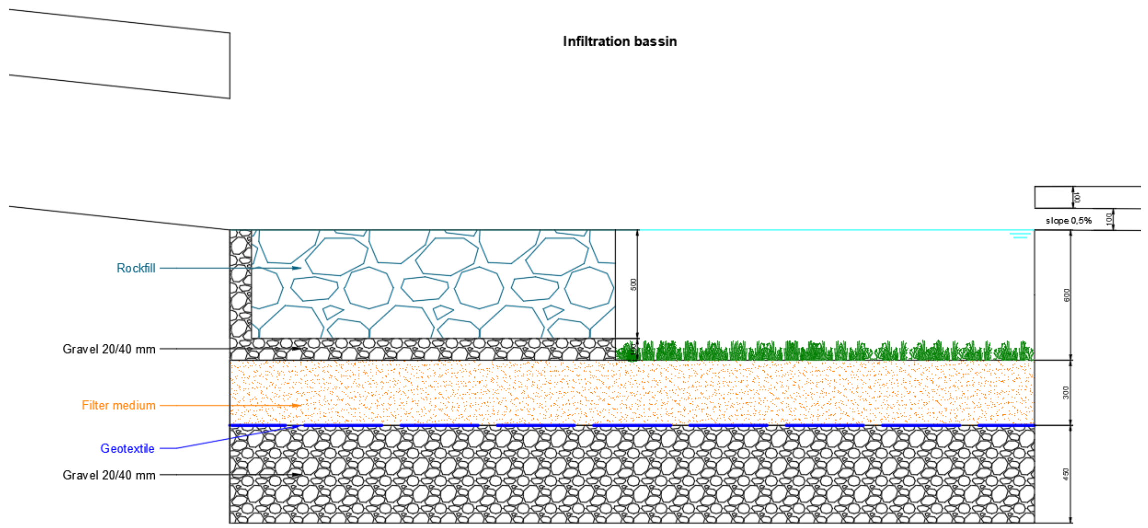

Figura 55. Sección tipo de las balsas de infiltración

La sección tipo estará compuestas por las siguientes capas:
- Superficie libre
- Capa de escollera
- Subbase de grava bajo escollera
- Medio filtrante
- Lámina geotextil
- Subbase de grava
- Geomembrana impermeable (si no infiltración)
- Tubo dren (si no infiltración)

La superficie libre disponible en las áreas de infiltración permite almacenar a modo de depósito una altura de agua de hasta 600mm, vaciándose por completo en un período de 48 horas tanto por su base como por sus laterales. Debido a ello, la infraestructura de entrada se colocará con la cota de solera a esta altura. Esta infraestructura se trata de una tubería cuyo diámetro varía si nos encontramos en la balsa de infiltración de la cuenca N1 o N2. Al igual que en los jardines de lluvia, se incluye una estructura de alivio, normalmente un aliviadero de tipo buzón, con un altura de 100 mm para dirigir la escorrentía aguas abajo en condiciones de seguridad, con una pendiente del 0,5% para los eventos que excedan al de diseño. Además, se propondrá también cierto resguardo entre la cota superior del aliviadero y la cota de acabado de la superficie adyacente de 100 mm. Además, la superficie libre estará cubierta con vegetación autóctona, con una densidad mínima de 16 ud/m2.

La entrada contará con un elemento disipador de energía compuesto por una capa de escollera de un diámetro medio de 250 mm. Su forma será trapezoidal y tendrá un espesor de 50 mm. Bajo la capa de escollera se colocará una capa de grava de 100 mm de espesor y gravas de tamaño 20/40 mm.

El medio filtrante tiene las siguientes características:
- El espesor será de 300 mm.
- La permeabilidad será de 250 mm/h
- La porosidad será del 0,5.
- El contenido en materia orgánica será entre 3-5%
- El pH estará entre 5.5 y 8.5
- La conductividad eléctrica será menor que 3300 microS/cm

Entre la medio filtrante y la capa de almacenamiento se colocará una lámina de geotextil no tejido para asegurarse que las partículas del suelo no pasan de una capa a otra. Esta lámina posee las mismas funciones que en los pavimentos permeables y las mismas características:
- Masa por unidad de superficie entre 125 y 160 gr/m2.
- Permeabilidad vertical entre 100 y 130 mm/s.

La subbase de grava debe ser más permeable que el medio filtrante.
- El espesor será de 450 mm
- El tamaño de árido se colocará entre 20 y 40 mm

12 PLAN DE MONITORIZACIÓN Y MANTENIMIENTO

Se ha determinado un plan de monitorización y mantenimiento de la solución óptima. El plan de monitorización se desarrolla para conocer el estado de la infraestructura y hacer un seguimiento de esta mientras que el plan de mantenimiento se ocupa de realizar inspecciones para determinar el nivel de eficiencia y planear las necesidades de mantenimiento, realizar operaciones y mantenimiento del sistema de drenaje, gestionar el paisaje y gestionar los residuos asociado con suelos o material contaminados, así como residuos generados por el mantenimiento. Estos planes se desarrollan con mayor contenido en el *Annex Nº6: Monitoring and maintenace program*

El plan de monitorización buscará realizar un seguimiento de la calidad y la cantidad de agua en tres puntos de monitorización diferentes:

Figure 1. Monitoring points

El seguimiento de la cantidad se realizará mediante caudalímetros ultrasónicos y sonda de presión hidrostática. La calidad se evaluará mediante los siguientes parámetros:

Tabla 19. Parámetros de análisis de calidad

Parámetro	Método de análisis	Intervalo
Sólidos Suspendidos Totales	UNE – EN 872	-
Demanda biológica de oxígeno a los 5 días	Respirometría	0 – 80 mg O2/L
Demanda química de oxígeno	ISO 15705	4 - 40 mg O2/L 10 - 150 mg O2/L 25 - 1500 mg O2/L
Turbidez	Turbidimetría	0,02 – 800 NTU
Nitrógeno total	ISO 11905 – 1	0,20 – 20 mg O2/L
Fósforo total	ISO 6878/1	0,01 – 5 mg P/L

En lo que respecta al plan de mantenimiento se han determinado las siguientes tareas indicando su tipología, frecuencia y componente al que afecta para cada sistema de drenaje sostenible.

Tabla 20. Propuesta de mantenimiento de pavimentos permeables

Tarea	Tipo	Frecuencia	Componente
Inspección inicial tras construcción	Inspección	Mensualmente durante los tres primeros meses	Superficie pavimento
Inspección ordinaria para detectar zonas colmatadas o crecimiento de vegetación no deseada	Inspección	Semestralmente y tras fuertes lluvias	Superficie pavimento
Realización de ensayo de permeabilidad para determinar la necesidad de acciones correctivas	Inspección	Anualmente, durante los tres primeros años	Superficie pavimento
Barrido en seco o aspiración estándar	Periódica	Semestralmente (primavera y otoño)	Superficie pavimento
Eliminar vegetación no deseada sobre pavimento y áreas adyacentes	Ocasional	Anualmente o cuando sea necesario	Superficie pavimento
Corrección de niveles de vegetación o tierra de superficies adyacentes que se hayan elevado hasta 50 mm del nivel del pavimento	Correctiva	Cada 5 años o cuando sea necesario	Superficie pavimento
Reparación de cualquier depresión, grieta o adoquín rotos que comprometa la capacidad estructural del pavimento o un riesgo para los usuarios y reemplazo del material de las juntas	Correctiva	Quinquenalmente o cuando sea necesario	Superficie pavimento
Rehabilitación de la superficie y la parte superior de la subbase mediante aspiración en profundidad, si se ha reducido la permeabilidad significativamente por la colmatación	Correctiva	Cada 15 años	Superficie pavimento
Reconstrucción al final de la vida útil	Correctiva	Al final de la vida útil (25 años)	Superficie pavimento

Tabla 21. Propuesta de mantenimiento de jardines de lluvia

Tarea	Tipo	Frecuencia	Componente
Retirar hojas, basura, sedimentos y hierbas no deseadas de la superficie de infiltración	Periódica	Mensualmente	Superficie, aliviadero y entrada
Eliminar los sedimentos acumulados en la entrada y aquellos que se queden atrapados en el aliviadero	Periódica	Anualmente o cuando sea necesario	Superficie, aliviadero y entrada
Riego para mantener la buena densidad de vegetación	Periódica	Mensualmente	Vegetación
Replantar áreas con poca vegetación	Correctiva	Cuando sea necesario	Vegetación
Revisión ordinaria para detectar la acumulación de sedimentos, encharcamientos, daños en la vegetación y erosión	Inspección	Semestralmente	Superficie, aliviadero, entrada y vegetación
Inspección técnica de las estructuras de entrada y de la superficie de infiltración (si procede)	Inspección	Semestralmente y después de fuertes lluvias	Superficie
Inspección de evaluación del tiempo de vaciado	Inspección	Semestralmente y después de fuertes lluvias	Superficie
Rellenar zonas erosionadas y mejorar la protección contra la erosión si fuese necesario	Correctiva	Cuando sea necesario	Superficie, aliviadero, entrada y vegetación
Realización de ensayo de permeabilidad al medio filtrante para determinar la necesidad de acciones correctivas	Inspección	Bienalmente	Medio filtrante
Eliminar acumulaciones de limo, escarificar la superficie y rellenar con sustrato	Ocasional	Cada 3 años o cuando sea necesario	Medio filtrante
Inspección técnica en busca de evidencia animal en el interior	Inspección	Anualmente	Medio filtrante
Realización de ensayo de permeabilidad para determinar la necesidad de acciones correctivas	Inspección	Anualmente, durante los tres primeros años	Superficie
Reconstrucción al final de la vida útil	Correctiva	Al final de la vida útil (30 años)	Superficie, vegetación, entrada, aliviadero, medio filtrante, gravas, dren

Tabla 22. Propuesta de mantenimiento de balsas de infiltración

Tarea	Tipo	Frecuencia	Componente
Retirar hojas, basura y hierbas no deseadas de la superficie de infiltración	Periódica	Mensualmente	Superficie, aliviadero y entrada
Eliminar los sedimentos acumulados en la entrada y aquellos que se queden atrapados en el aliviadero	Periódica	Anualmente o cuando sea necesario	Superficie, aliviadero y entrada
Inspección técnica de las estructuras de entrada y salida y aliviadero en busca de obstrucciones	Inspección	Semestralmente y después de fuertes lluvias	Superficie
Inspección de la superficie de infiltración	Inspección	Semestralmente y después de fuertes lluvias	Superficie
Riego para mantener la buena densidad de vegetación	Periódica	Mensualmente	Vegetación
Cortar la vegetación y retirar las malas hierbas	Periódica	Mensualmente	Vegetación
Replantar áreas con poca vegetación	Correctiva	Cuando sea necesario	Vegetación
Inspección de la balsa en búsqueda de charcos que evidencien zonas colmatadas	Inspección	Anualmente	Superficie
Revisión ordinaria para detectar la acumulación de sedimentos y erosión	Inspección	Anualmente	Superficie, aliviadero, entrada y vegetación
Verificación de que no se estén produciendo asentamientos	Inspección	Anualmente	Superficie
Inspección de evaluación del tiempo de vaciado	Inspección	Semestralmente y después de fuertes lluvias	Superficie
Rellenar zonas erosionadas y mejorar la protección contra la erosión si fuese necesario	Correctiva	Cuando sea necesario	Superficie, aliviadero, entrada y vegetación
Realización de ensayo de permeabilidad para determinar la necesidad de acciones correctivas	Inspección	Bienalmente	Superficie, gravas
Inspeccionar taludes y verificar topografía	Ocasional	Bienalmente	Superficie
Nivelar la base de la balsa y reinstalar los niveles establecidos en el diseño	Ocasional	Cuando sea necesario	Superficie
Reconstrucción al final de la vida útil	Correctiva	Al final de la vida útil (30 años)	Superficie, vegetación, entrada, aliviadero, medio filtrante, gravas, dren

13 VALORACIÓN ECONÓMICA

La valoración económica viene recogida en el *Documento Nº3: Economic assessment*., donde se determina que el presupuesto del proyecto asciende a la cifra de VEINTICUATRO MILLONES CUATROCIENTOS CINCUENTA Y NUEVE MIL CUATROCIENTOS TREINTA EUROS CON VEINTISIETE CÉNTIMOS.

14 OBJETIVOS DE DESARROLLO SOSTENIBLE. AGENDA 2030

La Organización de las Naciones Unidas aprobó en 2015 la Agenda 2030 sobre el Desarrollo sostenible, publicando los 17 Objetivos de Desarrollo Sostenible que permitirán el impulso del crecimiento económico, el compromiso con las necesidades sociales y la protección del medio ambiente.

Figura 56. Objetivos de Desarrollo Sostenible. Fuente: ONU

Los Objetivos de Desarrollo Sostenible identificados en la Agenda 2030 son:

- Objetivo 1. Poner fin a la pobreza en todas sus formas en todo el mundo
- Objetivo 2. Poner fin al hambre, lograr la seguridad alimentaria y la mejora de la nutrición y promover la agricultura sostenible.
- Objetivo 3. Garantizar una vida sana y promover el bienestar para todos en todas las edades
- Objetivo 4. Garantizar una educación inclusiva, equitativa y de calidad y promover oportunidades de aprendizaje durante toda la vida para todos.
- Objetivo 5. Lograr la igualdad entre los géneros y empoderar a todas las mujeres y las niñas.
- Objetivo 6. Garantizar la disponibilidad de agua y su gestión sostenible y el saneamiento para todos.
- Objetivo 7. Garantizar el acceso a una energía asequible, segura, sostenible y moderna para todos.
- Objetivo 8. Promover el crecimiento económico sostenido, inclusivo y sostenible, el empleo pleno y productivo y el trabajo decente para todos.
- Objetivo 9. Construir infraestructuras resilientes, promover la industrialización sostenible y fomentar la innovación.
- Objetivo 10. Reducir la desigualdad en y entre los países.
- Objetivo 11. Lograr que las ciudades y los asentamientos humanos sean inclusivos, seguros, resilientes y sostenibles.
- Objetivo 12. Garantizar modalidades de consumo y producción sostenibles.
- Objetivo 13. Adoptar medidas urgentes para combatir el cambio climático y sus efectos.
- Objetivo 14. Conservar y utilizar en forma sostenible los océanos, los mares y los recursos marinos para el desarrollo sostenible.
- Objetivo 15. Gestionar sosteniblemente los bosques, luchar contra la desertificación, detener e invertir la degradación de las tierras y detener la pérdida de biodiversidad.
- Objetivo 16. Promover sociedades, justas, pacíficas e inclusivas.
- Objetivo 17. Revitalizar la Alianza Mundial para el Desarrollo Sostenible

El presente trabajo final de máster, en un contexto en el que se busca el compromiso de comunidad científica en la mejora de la sociedad en el marco de los ODS, ha perseguido el cumplimiento de ciertos objetivos mencionados anteriormente. En particular, los objetivos buscados en este trabajo han sido:

- **Objetivo 6.** Garantizar la disponibilidad de agua y su gestión sostenible y el saneamiento para todos.
- **Objetivo 9.** Construir infraestructuras resilientes, promover la industrialización sostenible y fomentar la innovación.
- **Objetivo 11.** Lograr que las ciudades y los asentamientos humanos sean inclusivos, seguros, resilientes y sostenibles.
- **Objetivo 13.** Adoptar medidas urgentes para combatir el cambio climático y sus efectos.
- **Objetivo 15.** Gestionar sosteniblemente los bosques, luchar contra la desertificación, detener e invertir la degradación de las tierras y detener la pérdida de biodiversidad.

A continuación, se exponen las contribuciones del trabajo para la consecución de cada uno de los objetivos, mencionando las metas específicas que se han buscado alcanzar:

Objetivo 6. Garantizar la disponibilidad de agua y su gestión sostenible y el saneamiento para todos

Uno de los principales objetivos a los que contribuye el trabajo es a la gestión sostenible del agua mediante sistemas de drenaje sostenibles. En concreto, las metas indicadas por este objetivo que se buscan son la 6.3, 6.5 y 6.B.

6.3 De aquí a 2030, mejorar la calidad del agua reduciendo la contaminación, eliminando el vertimiento y minimizando la emisión de productos químicos y materiales peligrosos, reduciendo a la mitad el porcentaje de aguas residuales sin tratar y aumentando considerablemente el reciclado y la reutilización sin riesgos a nivel mundial.

6.5 De aquí a 2030, implementar la gestión integrada de los recursos hídricos a todos los niveles, incluso mediante la cooperación transfronteriza, según proceda.

6.B Apoyar y fortalecer la participación de las comunidades locales en la mejora de la gestión del agua y el saneamiento.

Objetivo 9. Construir infraestructuras resilientes, promover la industrialización sostenible y fomentar la innovación

Con una menor relación que el objetivo anterior, se busca en este objetivo el desarrollo e impulso de infraestructuras sostenibles como son los sistemas de drenaje sostenibles. Además, de cierta forma se menciona las áreas industriales que, aunque no se entre en detalle de desarrollo de actividades industriales sostenibles, se trata el desarrollo de medidas sostenibles en el ámbito de la industrialización. La meta a la que hace mención este trabajo es la 9.1.

9.1 Desarrollar infraestructuras fiables, sostenibles, resilientes y de calidad, incluidas infraestructuras regionales y transfronterizas, para apoyar el desarrollo económico y el bienestar humano, haciendo especial hincapié en el acceso asequible y equitativo para todos.

Objetivo 11. Lograr que las ciudades y los asentamientos humanos sean inclusivos, seguros, resilientes y sostenibles.

Este objetivo se persigue desde diferentes frentes, el primero es el que tiene relación con las inundaciones y las consecuencias ocasionadas por los desastres naturales, aunque también se trata el uso eficiente de recursos y la implementación de políticas relacionada con ellos. Las metas de este objetivo perseguidas en el trabajo son la 11.5 y 11.B.

11.5 De aquí a 2030, reducir significativamente el número de muertes causadas por los desastres, incluidos los relacionados con el agua, y de personas afectadas por ellos, y reducir considerablemente las pérdidas económicas directas provocadas por los desastres en comparación con el producto interno bruto mundial, haciendo especial hincapié en la protección de los pobres y las personas en situaciones de vulnerabilidad.

11.B De aquí a 2020, aumentar considerablemente el número de ciudades y asentamientos humanos que adoptan e implementan políticas y planes integrados para promover la inclusión, el uso eficiente de los recursos, la mitigación del cambio climático y la adaptación a él y la resiliencia ante los desastres, y desarrollar y poner en práctica, en consonancia con el Marco de Sendai para la Reducción del Riesgo de Desastres 2015-2030, la gestión integral de los riesgos de desastre a todos los niveles.

Objetivo 13. Adoptar medidas urgentes para combatir el cambio climático y sus efectos.

El cambio climático es uno de los mayores detonantes de las acciones preventivas y correctivas en ingeniería hidráulica, sobre todo con los eventos extraordinarios que cada vez son más frecuentes. Por ello se persigue este objetivo, ya no solo para mitigar los efectos que tienen las consecuencias del cambio climático, como son los eventos extremos, sino también tener en cuenta el cambio climático a la hora de la realización de proyectos y procurar instaurar prácticas sostenibles que consigan reducirlo. Las metas a las que se hace mención en este trabajo son la 13.1, 13.2 y 13.3.

13.1 Fortalecer la resiliencia y la capacidad de adaptación a los riesgos relacionados con el clima y los desastres naturales en todos los países.

13.2 Incorporar medidas relativas al cambio climático en las políticas, estrategias y planes nacionales.

13.3 Mejorar la educación, la sensibilización y la capacidad humana e institucional respecto de la mitigación del cambio climático, la adaptación a él, la reducción de sus efectos y la alerta temprana.

Objetivo 15. Gestionar sosteniblemente los bosques, luchar contra la desertificación, detener e invertir la degradación de las tierras y detener la pérdida de biodiversidad.

Trabajando en los beneficios de los sistemas de drenaje sostenibles, aparte de considerar las mejora en cuanto al control de cantidad y calidad de agua, se ha hecho mención a su carácter favorable en cuanto a los ecosistemas y la biodiversidad. Siendo así, la implantación de estos sistemas persigue las metas 15.3 y 15.A.

15.3 Para 2030, luchar contra la desertificación, rehabilitar las tierras y los suelos degradados, incluidas las tierras afectadas por la desertificación, la sequía y las inundaciones, y procurar lograr un mundo con una degradación neutra del suelo.

15.A Movilizar y aumentar de manera significativa los recursos financieros procedentes de todas las fuentes para conservar y utilizar de forma sostenible la diversidad biológica y los ecosistemas.

15 CONCLUSIÓN

En el presente trabajo se han evidenciado la satisfactoria adecuación de sistemas de drenaje sostenible (SuDS) en polígonos industriales, obteniendo beneficios que pueden llegar a ser incluso superiores a los producidos en zonas urbanas. Se han presentado estos diferentes beneficios en polígonos industriales, destacando:

- La **reducción de los volúmenes de escorrentía** y **laminación** de los hidrogramas, procurando la conservación hidrológica y ecológica de los medios receptores y la mitigación del riesgo de inundaciones locales.

- El **eficiente tratamiento de las cargas contaminantes** que arrastra el agua de lluvia, consiguiéndose así causar menores impactos sobre las masas de agua que reciben estos volúmenes.

- **Ahorro económico en el ciclo integral del agua** de una ciudad. Con ellos, se puede disminuir el tamaño de los colectores que forman las redes de drenaje urbanas, lo que conlleva a menores costes de construcción.

- Creación de muchos **servicios ecosistémicos**, que buscan favorecer y mejorar la vida de los ciudadanos y de los seres vivos que forman la biodiversidad local.

Asimismo, se ha hecho mención a la selección de los sistemas de drenaje más adecuados a la situación, evaluando diferentes factores y determinando la idoneidad de estos sistemas para encontrar la solución óptima. Todo ello apoyado por guías municipales que respaldan la elección de determinación del sistema que mejor se adapte al emplazamiento de estudio, así como otras guías internacionales que sirven de apoyo para el desarrollo e implantación de determinados SuDS.

En el caso concreto de estudio, se ha demostrado que los sistemas de drenaje sostenible son la solución óptima a la red de drenaje del polígono industrial de Quart de Poblet, Valencia, España. Esta afirmación se ha basado en una comparativa de cuatro alternativas diferentes, variando enfoques tradicionales y de desarrollo sostenible y en la simulación mediante herramientas de modelización hidráulicas. Se ha realizado un análisis multicriterio que tiene en cuenta aspectos financieros, hidráulicos, energéticos, medioambientales y sociales demostrando que, aunque estás alternativas sean más costosas económicamente, su respuesta hidráulica es más que satisfactoria, llegando a acercarse a objetivos hidráulicos ambiciosos y que, por supuesto, suponen una mejora en cuanto a ahorro energético, emisión de gases efecto invernadero y del bienestar social y medioambiental general.

A la vista de todo el trabajo desarrollado, no cabe duda de que los sistemas de drenaje sostenibles son una solución que debería estudiarse como alternativa para los nuevos proyectos de sistema de drenaje o, como mínimo, considerar integrarlos en sistemas de drenaje ya existentes, teniendo en cuenta su particular eficacia en polígonos industriales.

No obstante, pese a todos los resultados favorables en este y otros estudios del mismo ámbito, todavía queda mucho trabajo por delante para conseguir una implantación de los SuDS de manera generalizada y estandarizada. Para conseguir esto se deberá trabajar conjuntamente en dos aspectos fundamentales: la financiación y la reglamentación.

- **Financiación.** La apuesta por estas técnicas de drenaje sostenibles no suele requerir recursos financieros adicionales, aunque sí una gestión más efectiva de los fondos existentes. El análisis de su rentabilidad no puede basarse únicamente en criterios económicos y resultados hidrológicos obtenidos, sino que debería evaluar también los beneficios que aportan en calidad del paisaje, adaptación y mitigación del cambio climático, y mejoras en el bienestar de la población transformándolos a valor monetario.

- **Reglamentación.** Es importante contar con un respaldo jurídico para las iniciativas relacionadas con la gestión sostenible del agua. Así pues, hay que optar por promover estas técnicas sostenibles de forma eficiente a través de los marcos existentes, para que se consideren una auténtica alternativa de drenaje.

En adición, se debe trabajar para mejorar dos parámetros: la **colaboración intersectorial**, cuya mejora facilitará el cambio y la toma de decisiones, y la **base de conocimientos,** dando a conocer los procesos y beneficios de los SuDS mediante campañas de formación y concienciación, formando a profesionales para el diseño, ejecución, mantenimiento y evaluación y desarrollando criterios generales comunes para poder diferentes opciones para la gestión de los recursos hídricos.

DOCUMENT 1. REPORT

SUSTAINABLE DRAINAGE SYSTEMS (SUDS) IN INDUSTRIAL AREAS:
APPLICATION TO THE INDUSTRIAL AREA IN QUART DE POBLET (VALENCIA)

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MÁSTER UNIVERSITARIO EN INGENIERÍA DE CAMINOS, CANALES Y PUERTOS
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UNIVERSIDAD POLITÉNICA DE VALENCIA

UNIVERSITAT POLITÈCNICA DE VALÈNCIA

Summary

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1 OBJECT

The present document is described as the Final Master Thesis of the student Ana Álvarez Pérez in order to obtain the Máster Universitario de Ingeniería de Caminos, Canales y Puertos at the Universidad Politécnica de Valencia (UPV) as well as the engineering diploma at l'École Nationale des Ponts et Chaussées (ENPC) within the context of the double-diploma agreement.

The final Master Thesis, hereafter called TFM, concerns the integration of SuDS in industrial areas. This work is presented as a solution to drainage systems in industrial areas in general, including its professional application to the industrial area of Quart de Poblet, Valencia.

The tutoring of the work has been guided by Dr. Ignacio Andrés Doménech, professor at the Universidad Politécnica de Valencia, member of the Department of Hydraulic and Enivronment Engineering and researcher at the Research Institute of Water and Environmental Engineering (IIAMA). In addition, the work is guided by a cotutor, Dr. Damien Tedoldi, researcher specialised in urban hydrology at l'Ecole Nationale des Ponts et Chaussées and professor at the Institut National des Sciences Appliquées in Lyon.

The organisation of the project is presented below, divided into the following documents:

- DOCUMENT Nº1: REPORT
  - Annex Nº1: Technical framework. Sustainable drainage systems
  - Annex Nº2: Initial data and design conditions
  - Annex Nº3: Solutions proposal
  - Annex Nº4: Comparative multi-criterial analysis
  - Annex Nº5: Development of the optimal solution
  - Annex Nº6: Monitoring and maintenance program
  - Annex Nº7: References

- DOCUMENT Nº2: MAPS

- DOCUMENT Nº3: ECONOMIC ASSESSMENT

2 BACKGROUND

The need for the implementation of efficient drainage systems is becoming increasingly present. This need is largely determined by the development of cities and urbanised areas. The development of cities as society progresses implies an increase in soil impermeability, and this impermeability has direct and indirect negative effects on various aspects. The main problem is the alteration of the intrinsic natural characteristics of the soil. Among the negative aspects affected by the alteration of the soil's natural characteristics we can find:

- Decrease in the capacity for retention and filtration of rainwater.
- Inertisation and desertification of the soil surface.
- Increase in ambient temperature in cities
- Deterioration of atmospheric quality
- Landscape alteration
- Denaturalisation of the environment

The overpopulation of cities is reflected in data provided by the World Bank. More than 4 billion people worldwide, representing more than 50% of the world's population, live in cities. This growth trend is set to continue with prospects of a doubling of the urban population by 2050, with almost 7 out of 10 people living in cities.

This development of the polis affects the warming of the atmosphere. Among various aspects popularly known as the gas pollution from vehicles, it is added, focusing on the subject that concerns us, the reverberation of the heat that adheres to permeable surfaces such as pavements or asphalt. This phenomenon significantly increases the ambient temperature; including that, in cities like Valencia, the temperature of pavements can reach close to 90 degrees Celsius.

However, the greatest consequence of impermeabilisation is given by the modification of the water resources cycle; where a change causes the modification of the natural path of water, leading to an increase in:

- Runoff volumes
- Peak flows
- Increased flows, being lower in dry periods and higher in rainfall periods.
- Pollution

The conventional system focuses on the collection of rainwater through sewerage networks. A methodology that tries to evacuate the water as quickly as possible by taking it out of the area of action and, if necessary, to be treated in wastewater treatment plants. A system such as this, faces a problem when its capacity is lower than what is required. Large quantities of rainwater to be treated means polluted water discharges, reduced performance of the treatment plants or even disasters such as local flooding. It can be seen that until now, traditional drainage systems have focused more on the quantity of water to be managed, ignoring other aspects such as quality or the service that can be given to this water.

In this context, sustainable drainage systems (SuDS) come into the picture. SuDS are the solution to a problem that affects many areas of the world. The solution lies in a change in the management of rainwater; instead of trying to act at the end of the process, the aim is to try to ensure that the water is treated at source. This would impose exclusively hydrological measures (study, planning and quantification of the water resource) instead of hydraulic measures (design and operation of civil works for the use of the resource).

SuDS are presented as a solution with a smart and efficient water management where the main objective is not to collect water, transport it and treat it, but to try to treat it as quickly as possible thanks to infiltration. Decentralised retention allows water to be infiltrated and retained at source to be progressively released at peak times and used for non-potable water uses such as irrigation.

3 LEGISLATIVE FRAMEWORK

The regulatory context of drainage systems, with particular reference to sustainable drainage systems, is parameterised by the principles of governance of European and Spanish legislation, these being the improvement of the state of water masses, protection against floods and droughts, adaptation and mitigation of the impact of climate change, reduction of energy consumption in the urban water cycle, conservation of biodiversity and reinforcement of the values of water and green spaces dedicated to the citizens.

This section has compiled the current European, national, regional and municipal regulations governing sustainable drainage systems.

3.1 European legislation

- **Directive 2000/60/CE** of the European Parliament and of the Council, of 23 October 2000, establishing a framework for Community action in the field of water policy. It was created in response to the need to unify actions in the field of water management in the European Union, taking measures to protect water masses both in qualitative and quantitative terms and thus guaranteeing their sustainability, and setting deadlines for the implementation of the directive in the different EU Member States, as well as for the achievement of objectives.

- **Directive 2007/60/CE** of the European Parliament and of the Council of 23 October 2007 on the assessment and management of flood risks.

3.2 National legislation

- **Real Decreto 1/2016**, of 8 January, approving the revision of the Hydrological Plans of the Western Cantabrian, Guadalquivir, Ceuta, Melilla, Segura and Júcar river basin districts, and of the Spanish part of the Eastern Cantabrian, Miño-Sil, Duero, Tajo, Guadiana and Ebro river basin districts. Article 44 states that new urbanisations, industrial estates and urban developments that may cause alterations in the drainage of the intercepted basin or basins must introduce sustainable drainage systems (use of permeable paving, stormwater tanks, etc...) that guarantee that the eventual increase in runoff with respect to the value corresponding to the pre-existing situation can be compensated for or is irrelevant.

- **Real Decreto 638/2016**, of 9 December, which modifies the Public Hydraulic Domain Regulations approved by Real Decreto 849/1986, of 11 April, the Hydrological Planning Regulations, appobed by Real Decreto 907/2007, of 6 July, and other regulations on flood risk management, ecological flows, hydrological reserves and wastewater discharge. The article 126 ter specifies that new urbanisations, industrial estates and urban developments in general must introduce sustainable drainage systems, such as permeable surfaces, so that the eventual increase in flood risk is mitigated. To this end, the urban development dossier must include a hydrological-hydraulic study that justifies it.

3.3 Autonomic legislation

- **Decreto 201/2015**, of 29 October, of the Consell, approved the Territorial Action Plan on Flood Risk Prevention in the Comunidad Valenciana (PATRICOVA). This Plan defends the management of Green Infrastructure against floods. Green Infrastructure is defined as the basic territorial system formed by the spaces of greatest environmental, cultural, landscape and visual value, the critical areas of the territory due to their susceptibility to risks, as well as their ecological and functional connections. Although it is considered a flood prevention plan as a result of the overflowing of watercourses or a rise in the water level, it incorporates the recommendation for the implementation of SUDS due to their contribution, among others, to the reduction of damage caused by short return period rainfall. In Title IV "On Defense Actions", Article 23, it is stated that in the design of Green Infrastructure, the use of Sustainable Urban Drainage Systems will be encouraged. Furthermore, it is added that the use of Sustainable Urban Drainage Systems will be promoted in all the municipalities of the Comunidad Valenciana.

- **Resolución 997/IX**, of 30 May 2017, by the Presidency of Les Corts de la Generalitat Valenciana, on the Incorporation of preventive and technical measures related to the use of SuDS in green infrastructure includes: in compliance with the provisions of Article 95. 1 of the Regulations of Les Corts, it is ordered to publish in the Butlletí Oficial de les Corts the Resolución 997/IX, on the incorporation of prevention measures and techniques related to the use of sustainable drainage systems in the design of green infrastructure included or associated with land use plans, approved by the Comisión de Obras Públicas, Infraestructuras y Transportes at the meeting of 30 May 2017".

3.4 Municipal legislation

- **Environmental Protection Ordinance on Sewage and Discharges to the Municipal Network**, *Ordenanza de Protección del Medio Ambiente relativa al Alcantarillado y Vertidos a la Red Municipal*, published by Quart de Poblet Townhall, definitively approved on 29 December 2016, including in chapter 2, point 12 that the discharge of this clean water should promote sustainable, low-impact environmental solutions, such as its diversion to natural watercourses or passive infiltration in open areas.

- **Integrated Sustainable Urban Development Strategy (EDUSI)**, *Estrategia de Desarrollo Urbano Sostenible Integrado*, of Quart de Poblet, co-financed by the FEDER Operational Programme for sustainable growth 2014- 2020, called by Order HAP/2427/2015 of 13 November as an action aimed at the improvement and rehabilitation of the urban environment.

4 PROJECT’S SCOPE

This work has two main objectives. The first is to carry out the implementation of sustainable drainage systems that can be integrated in industrial areas, identifying in a justified way those typologies that are best suited to this type of environment. While sustainable drainage systems are innovative in themselves, their application in industrial areas has not been studied in detail and few documents refer to the benefits of these systems in industrial estates where impermeability and pollutant content make the condition of runoff water very prejudicial to the receiving environment, both in quantity and quality.

Secondly, the optimal solution for the efficient treatment of rainwater in the industrial area of Quart de Poblet, Valencia, Spain will be determined, considering financial, hydraulic, environmental, energy and social criteria. With this objective in mind, the aim will be to ensure that, by means of the drainage system, the same flow is discharged into the receiving environment as in its natural state, previous to the urbanisation of the industrial estate. To achieve this objective, an analysis of the physical and administrative conditioning factors of the study area and its design parameters will be carried out to determine the drainage systems that best suits and resolves the problems in the industrial area.

By means of recommendation guides, a design of the drainage systems will be carried out, each one in accordance with its specifications. Subsequently, a verification will be carried out with the SWMM programme to check that each solution is correctly designed on the basis of the hydraulic variables determined and, finally, the optimum solution will be determined by means of a multi-criteria analysis using the E2Stormed tool.

In this way, this work will not only indicate the drainage systems to be implemented that best solve the problems of the application, but will also analyse the multiple benefits of SuDS, specifically in industrial areas, and present the need for regulatory framework and impulse from public and private organisations to invest more resources in these systems.

5 LOCATION

The industrial estate referred to in this study is located in Quart de Poblet, a municipality of Comunidad Valenciana, Spain, belonging to the province of Valencia.

Situated in Huerta Oeste, the area of the municipality is crossed by the Turia river to the north. To the west flows the Poyo basin, a seasonal ravine whose basin is located between the Turia and Jucar rivers and the Picassent ravine.

Figure 1. Location at state level. Source: GVA map viewer

Figure 2. Location at autonomic level. Source: GVA map viewer

Figure 3. Location at municipal level. Source: GVA map viewer

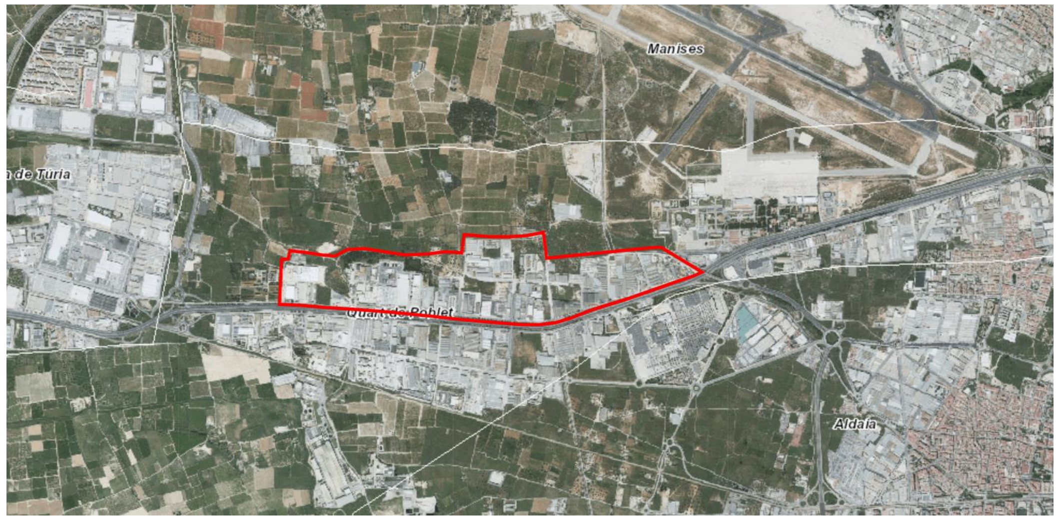

Figure 4. Location inside Quart de Poblet. Source: GVA map viewer

The main road traffic routes to the west of the metropolitan area of Valencia pass through the municipality of Quart de Poblet, which means that Quart de Poblet is very well integrated into the transport infrastructure network of the Valencian metropolitan area.

Thus, both the town centre and the numerous industrial estates are well connected by highway to the surrounding municipalities and other high-capacity roads, facilitating road transport, as well as public railway transport connections.

The main traffic axes in the study area are:

- A-3/E-901 Madrid-Valencia highway, which runs longitudinally to the south of the municipality and provides three accesses.
- V-30 highway ring road, located parallel to the Turia river, running diagonally from the northwest to the southeast.
- V-11 highway, connecting Quart de Poblet and Valencia airport.
- CV-408 and CV-31, local and provincial roads that serve as a link with neighbouring towns, passing through the industrial facilities.
- Metro lines 3, 5 and 9, which were adapted to replace the old railway station on the C-4 Valencia-North - Ribarroja del Turia line.
- Metrobus lines connecting Quart de Poblet with Valencia and other localities; 106 (Torrent), 150 (Manises Airport), 161 (Valencia).

6 CURRENT SITUATION AND PROBLEMATIC

6.1 Poyo basin

The basin in which the project area is located is called the Poyo basin and is made up of the Poyo, Pozalet and La Saleta ravines. This basin is located between the Turia and Jucar rivers and covers an area of 450 km2.

The problems related to high-density rainfall, added to the considerable increase in urban development in the urban areas of the basin, have caused the ravines to contain very high flows at the entrance to the urban areas, causing a risk of flooding in several municipalities. In addition, as the Albufera lake is the current end point of the water, there is also the risk of sediment contribution to the Albufera.

Figure 5. Location of the Poyo basin. Source: F.Franch et al., 2010

The industrial estate is located downstream of the Pozalet ravine and upstream of La Saleta ravine, which is closer to the study area. The weakest point in the area is between Loriguilla and the A7 highway by-pass, with a drainage capacity much lower than that of the watercourse itself. Between Loriguilla and the A3 highway, the riverbed is no longer defined, causing an area of lamination where the water flows uncontrolled, affecting the adjoining industrial estates, one of which is the industrial estate under study.

Although the Poyo ravine is not an outlet for water from the industrial estate, it is important to know its hydrographical behaviour because, in situations of rainfall of a certain magnitude, part of the overflows of the Poyo ravine, downstream of the A7, flow northwards, increasing the flows of the Pozalet ravine and consequently of La Saleta ravine.

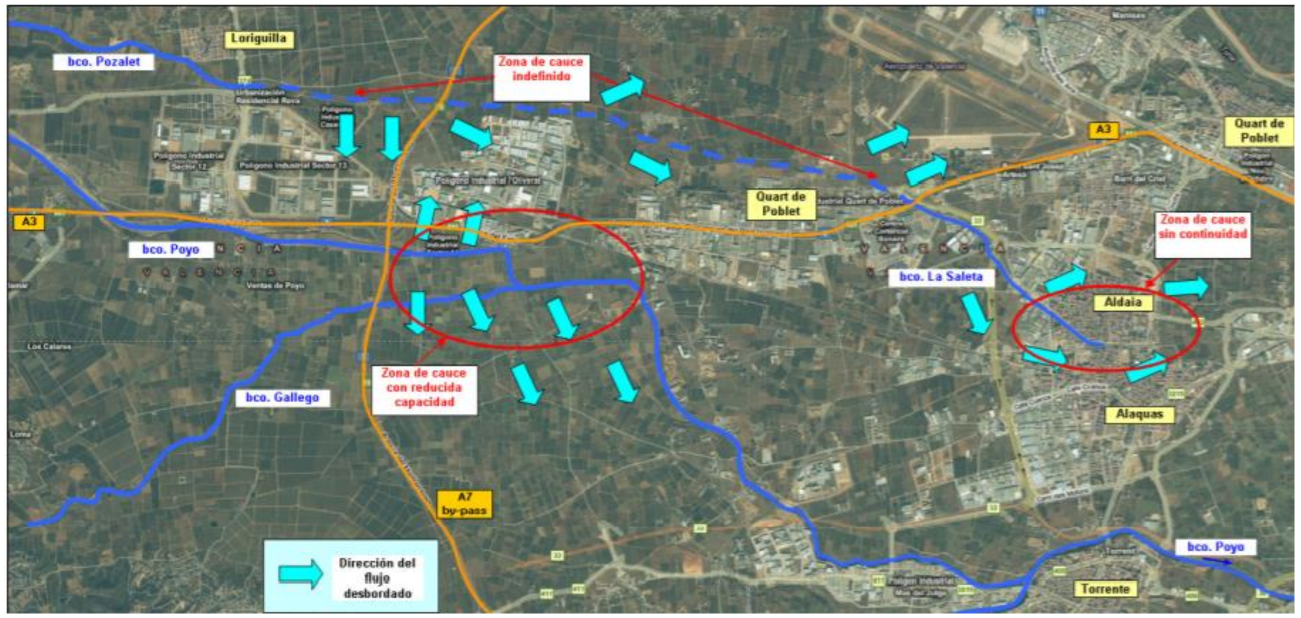

Figure 6. Hydraulic behaviour of the Pozalet-Saleta ravine. Source: F.Franch et al., 2010

At present, La Saleta ravine has enough capacity in its initial section to carry a flow of around 100 m3/s. This capacity is progressively decreasing until it is completely lost in the centre of Aldaia, which makes this town the most affected in terms of flood risk. The Pozalet ravine further complicates the situation of the towns linked to La Saleta, as La Saleta receives the waters of Pozalet, which means that the peak flow in flood situations increases considerably. An example of an overflow situation, which was catastrophic, was the flood of the year 2000 with a return period of 500 years.

In the following image, a result of the flood simulation for a return period of 500 years has been obtained to show the consequences visually.

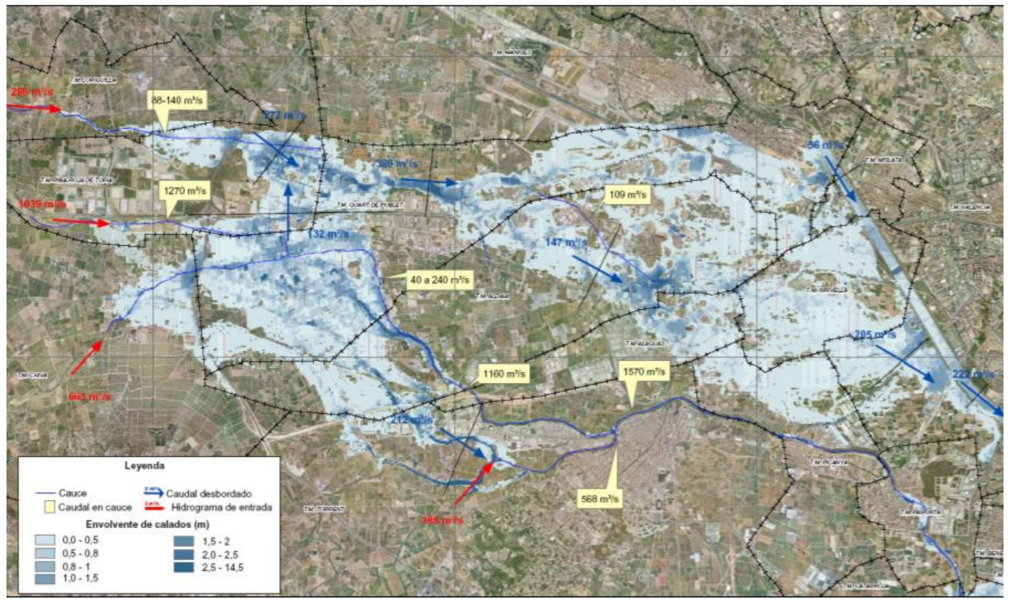

Figure 7. Flood simulation with representative event T=500 years (flood of 2000). Source: F.Franch et al., 2010

The Jucar Water Authority, *Confederación Hidrográfica del Jucar*, (CHJ) has presented different solutions to avoid the problem of overflowing of the riverbed and flooding. One of them is the plan to build a closed connection through the industrial estate in the northern part of Aldaia, diverting part of it through the drainage culvert of the CV-33 highway and creating a connection between La Saleta ravine and the Turia river.

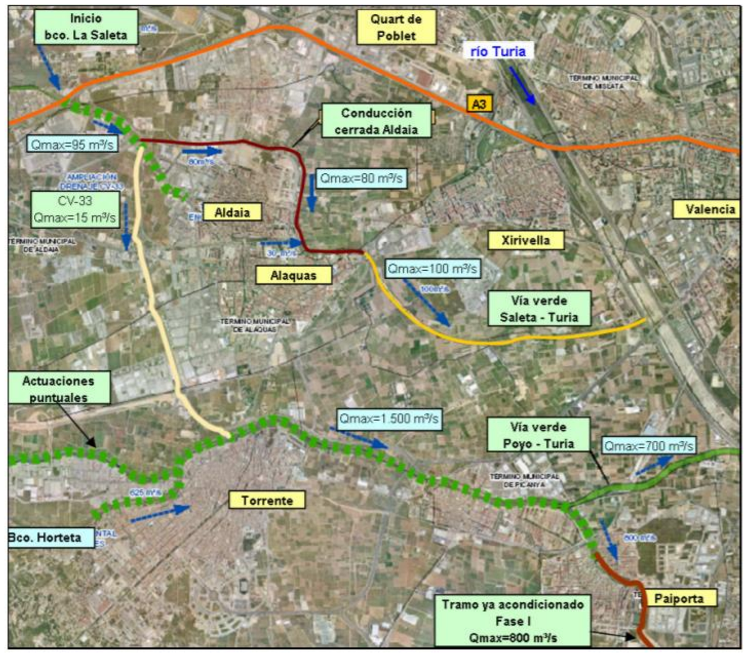

Figure 8. Solution CHJ ravine La Saleta. Source: F.Franch et al., 2010

6.2 Industrial área in Quart de Poblet

This work aims to solve the problem on a reduced scale, focusing on the industrial estate in Quart de Poblet and its contributions to La Saleta ravine. Currently, the industrial estate under study does not have a drainage system, so all the rainwater runoff runs freely through the streets and diverges into La Saleta ravine. This situation is of concern to the CHJ, which sees La Saleta ravine as being compromised due to the large amount of water it carries. The urban development of the areas surrounding the ravine has caused a considerable increase in runoff, pushing it to its limits, favouring the overflowing of the riverbed and the risk of flooding in the neighbouring villages.

In view of this situation, the CHJ requires Quart de Poblet townhall to solve the problems related to the amount of water discharged, encouraging it not only to build a drainage system that manages to control the channelling of rainwater, but also to reduce the amount of water discharged into La Saleta ravine. Ideally, the ambitious objective is to achieve the same flow into La Saleta ravine as that which was discharged previous to the urbanisation of the industrial estate. . For this reason, and always bearing in mind that we are working in an industrial area where the limitations due to pollutants play a major role, we have been driven to look for an alternative to the possible conventional drainage system to be able to laminate the runoff water and control the pollutants discharged.

To get an idea of the characteristics of the industrial estate, a series of photographs taken recently of the study area are shown below, which show the lack of drainage systems in the area and the condition of the roads in the industrial estate. The photographs correspond to a few hours after a rainfall event and show several areas where water remains ponded on the surface due to the absence of a drainage system.

The photographs also show the primitive condition of the roads, made up of unpaved paths and, in the best of cases, concrete and gravel paving. There are minor exceptions, such as the most easterly street of the industrial estate, where it has been possible to see that work has begun to improve the roads by asphalting them with bituminous asphalt binders.


Figure 9. Road status Calle Polígono 13, catchment N2


Figure 10. Road status Camí de la Canyada, catchment N1


Figure 11. Road status Calle Polígono 13, catchmentN1


Figure 12. Road status de la Pinadeta, catchment N3


Figure 13. Green area status Calle Polígono 13, catchment N1,

Figure 14. Road status Calle Canal Xuquer-Turia, catchment N3

Figure 15. Beginning of the improvement of Calle Sequia de Mislata, catchment N1

7 INITIAL DATA AND DESIGN CONDITIONS

The relevant aspects and parameters of the study area have been determined, such as urban planning, topography, geology, hydrogeology, geotechnics, pluviometry and the natural state of the catchment areas previous to urbanisation, in order to know their behaviour and the objective parameters that will help to design the different solutions.

Everything mentioned hereafter can be found explained in detail and in greater depth in *Annex Nº2: Initial data and design conditions.*

7.1 Urban and territorial planning

The urban and territorial planning of the study area is carried out by means of the General Urban Development Plan (PGOU) of Quart de Poblet, published in 2002. Based on the PGOU, which identifies the subcatchments for private use and public facilities corresponding to streets and green areas, the area is divided into three main land uses corresponding to the hydrological response of each one and the amount of runoff they generate.

Figure 16. Land uses division based on PGOU

The study area is thus divided into private subcatchments in pink, streets subcatchment in blue and green areas subcatchments in green.

7.2 Topography

Topography is a major conditioning factor in the design of possible solutions. Determining the slopes of the site is important in order to be able to determine the natural course of surface runoff and thus know the volume of water to be treated on each surface and better design the drainage system. In addition, slopes are a determining factor, because certain sustainable drainage systems are limited by the slope of the site.

The following figure shows the plan of the longitudinal slopes of each street subcatchment obtained after processing the information from the cartographic maps. This information can also be found to scale in *Document Nº2: Maps*.

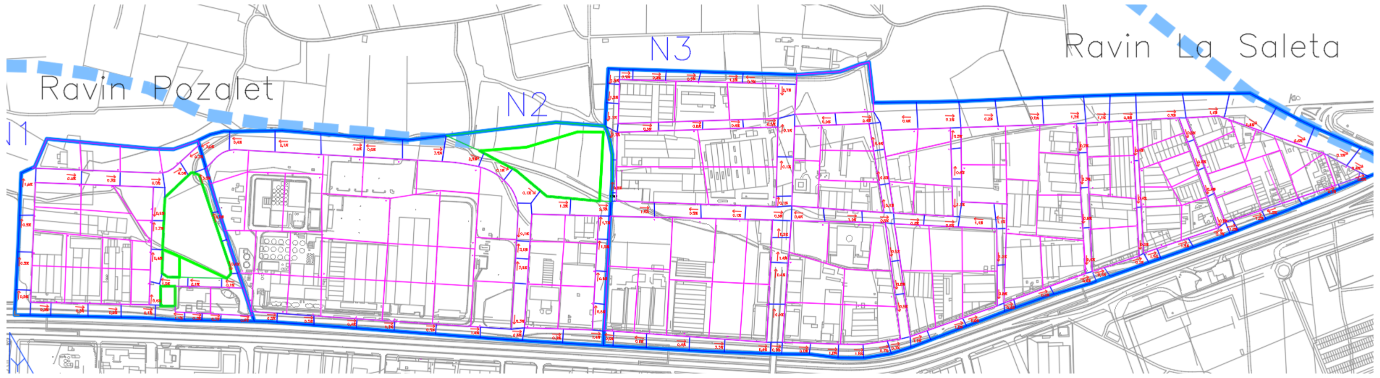

Figure 17. Longitudinal slopes on the study area

7.3 Geology and geotechnics

The study area, which belongs to the synclinal axis of the Cadena Ibérica, was formed during the Miocene as a consequence of the shortening of the area occupied by the Mesozoic sea, which produced the emersion of areas of positive relief and depressed areas aligned along Iberian axes. The Valencian region is located in one of these depressions, which was invaded by the sea in the Lower Tertiary period. This sea deposited sandy materials, initially conglomerates, forming a gulf of larger dimensions than the present one. In the Upper Tertiary, the sea regressed, creating a lagoon-like area, in which marl and limestone were deposited, which slowly filled in due to the contributions of water flows from the surrounding areas. Subsequently, the generation of a more extreme climate than the present one and the hydrological system gave rise to quaternary deposits from the erosion of the mountains upstream, while the coastline retreated, abandoning the old littoral sediments.

Regarding stratigraphy, the study area is located in the soils of the **Quaternary period**, specifically in the continental deposits corresponding to the **Lower Pleistocene with calcareous crust** and to the **Upper Pleistocene with modern stream mantles**, being red sandy clays with crustal edges.

The tectonics at the project site are presented as a wide morphological depression of complex tectonic origin. This depression, geographically known as the Huerta de Valencia, extends between the sea and the foothills of the Chiva and Buñol reliefs and is flanked to the north by the Náquera reliefs, and to the south by the Cullera-Alginet reliefs. The depressed area represents a synclinal axis of the Cadena Ibérica, in which materials have been deposited clearly after the main moment of compression which created the "Iberian" structures. On the other hand, this tectonic depression has been affected by later movements causally related to the tectonics of the "Betic" areas located further south, which in turn must be responsible for the more recent movements of the Mediterranean coasts.

7.4 Hidrogeology

7.4.1 Flood hazard

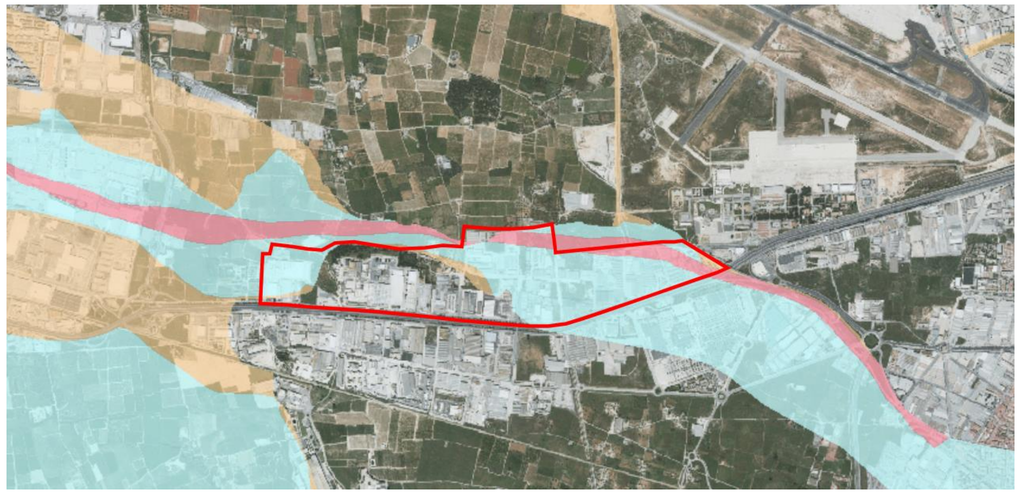
Figure 18. Flood hazard map by PATRICOVA. Source: GVA map viewer

The northern part of the industrial estate is considered a **level 1 hazard** zone, identified in pink on the map, with a high return period frequency of 25-year return period and a depth exceeding 0.8 meters. The whole of the N3 catchment area and a large part of the N2 is classified as a **level 3 hazard** zone, identified in light blue, with a high frequency of 25-year return period and a low depth of less than 0.8 meters. In terms of geomorphological hazard, identified in orange, the study area is characterized by a small **geomorphological hazard** zone to the east and in the center of the industrial estate.

7.4.2 Flood risk

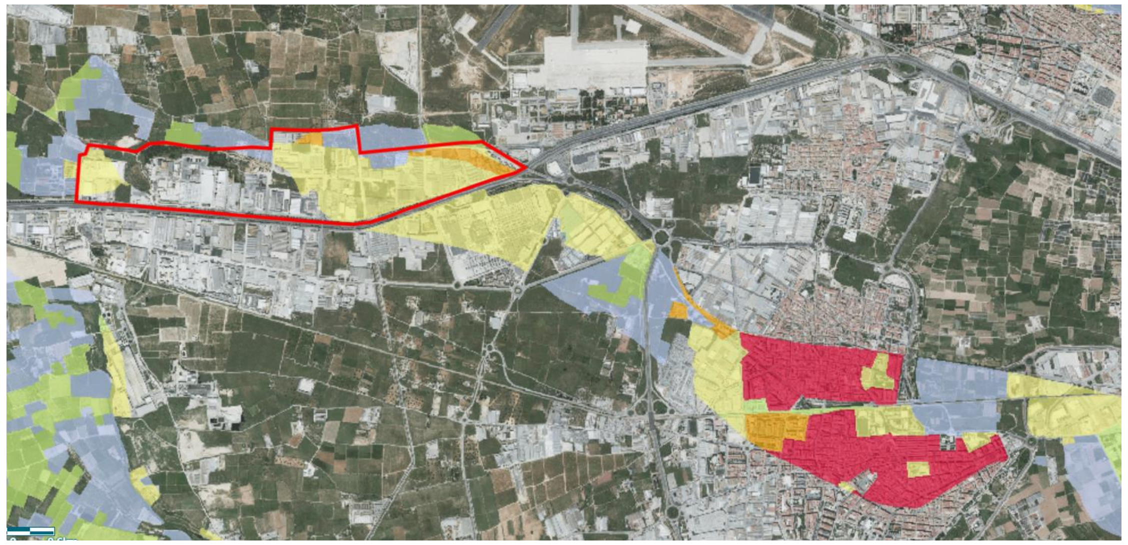
Figure 19. Flood risk map by PATRICOVA. Source: GVA map viewer

The flood risk map shows that the project area is an area where the risk is generally **medium**, in yellow, containing some areas of **low** risk, in green, and **very low** risk, in blue, and, to a lesser extent, areas of **high** risk, in orange. As indicated in the problem of runoff water discharges, the area close to the industrial estate is the municipality of Aldaia with a very high risk of flooding, identified in red.

7.5 Permeability

Figure 20. Permeability map. Source. GVA map viewer

The entire project area is classified as a **high permeability** zone in light green, being bordered to the east by a very high permeability zone in dark green. The coefficient of permeability of the soil for the design phase is determined to be:

$$Kc = \mathbf{12{,}9\ mm/h} = 3{,}58 \cdot 10^{-6}\ m/s$$

7.6 Return period

The design return period considered has been the one indicated in the Territorial Action Plan on Flood Risk Prevention of the Valencian Community (PATRICOVA, 2015). Corresponding to:

$$\boldsymbol{Tr = 15\ a\tilde{n}os}$$

7.7 Pluviometric analysis

The pluviometric analysis was carried out on the basis of the data obtained from the Valencia - Airport pluviometric station, which collects data from the historical series from 1966 to 2019, corresponding to 54 years of data collection. Using this data, a volumetric frequency analysis and an extreme analysis of the maximum daily rainfall for different return periods was carried out to obtain the design rainfall that will be used for the design and modelling of the case study.

7.7.1 Volumetric frequency analysis

The average monthly rainfall and the average number of rainy days per month for the available series is as follows:

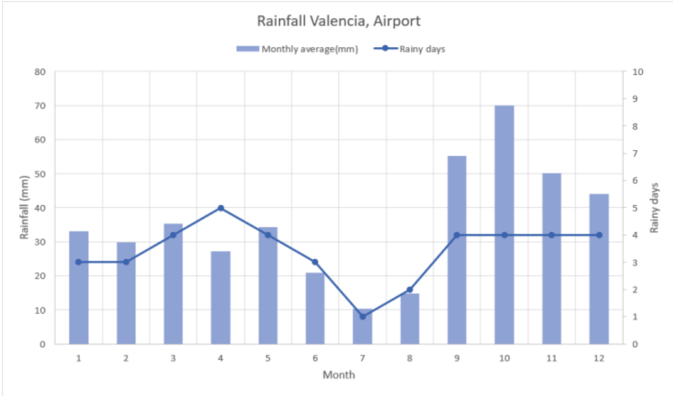

Figure 21. Diagram on average monthly rainfall and average number of rainy days

Using the pluviometric data it is possible to identify the sample daily rainfall distribution function representing the probability of non-exceedance for each daily rainfall volume, identifying the leading percentiles.

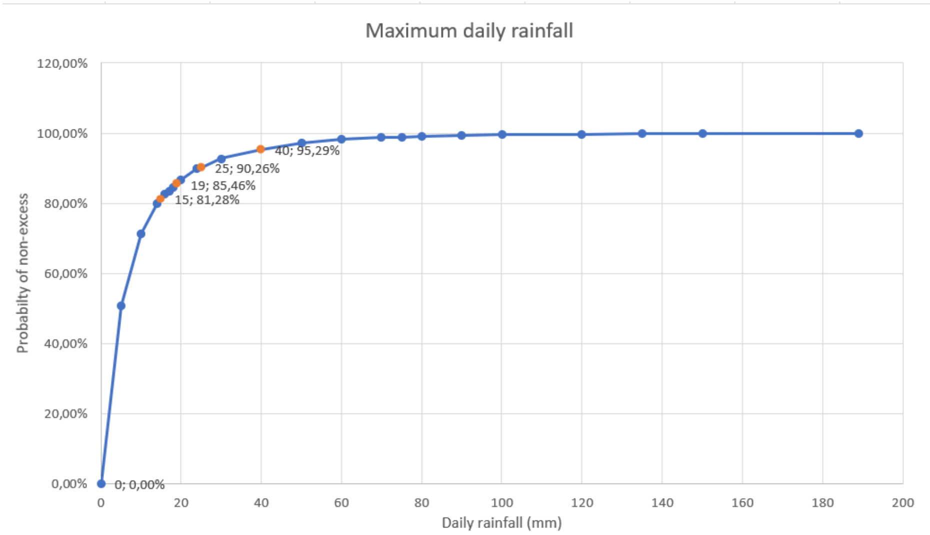

Figure 22. Graphic representation on probability of non-exceedance daily rainfall

From these results we can analyse:
- On 80% of rainy days, rainfall does not exceed 15 mm.
- On 85% of rainy days, rainfall does not exceed 19 mm.
- On 90% of the rainy days, rainfall does not exceed 25 mm.
- On 95% of rainy days, rainfall does not exceed 40 mm.

7.7.2 Extreme analysis

Extreme events are determined in order to design a drainage system that can function satisfactorily without producing local flooding that could cause material and environmental damage.

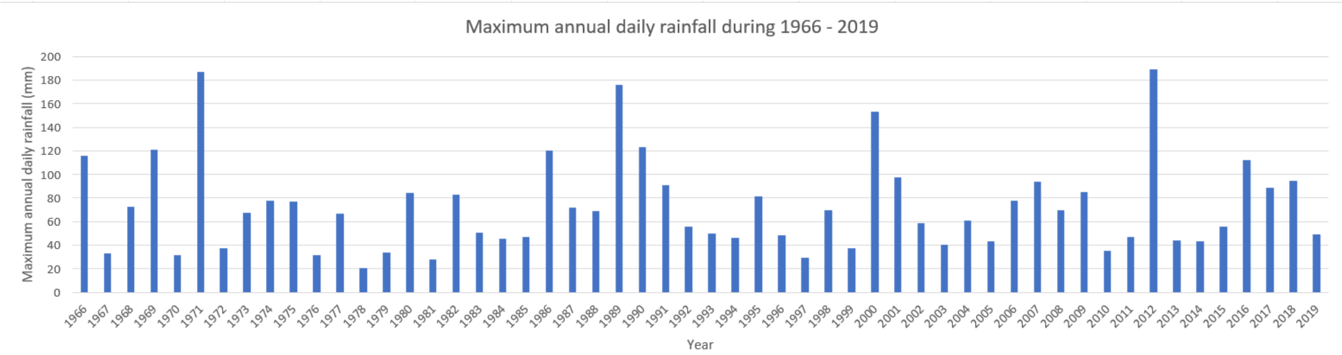

Figure 23. Maximum daily rainfall during 1966 – 2019 period

By means of the statistical analysis of the series studied, it has been possible to identify the theoretical distribution that most closely approximates the series analysed using the maximum likelihood method, this being the **SQRT-et-MAX distribution**, the representation of which is as follows:

$$F_x(x) = e^{\left(-k*(1+\sqrt{\alpha x})*e^{(-\sqrt{\alpha x})}\right)}$$

For the series data the parameter values are:

Table 1. Parameters of SQRT-et-MAX distribution for observed series

Parameter	Value
k	26,2913
α	0,49285

This identifies the maximum daily precipitation of the function for each return period. For the case study where the design return period has been determined to be 15 years, we obtain a maximum daily rainfall of **135,02 mm.**

Table 2. Maximum daily rainfall for each return period

Return period (years)	Maximum daily rainfall (mm)
2	61,60
5	94,56
10	119,70
15	135,02
25	155,15
50	184,08

7.7.3 Design storms

After the pluviometric analysis, the design storms corresponding to the IDF curve were determined, which estimate the maximum rainfall intensity corresponding to the time of concentration of the basin for a given return period and the design storms corresponding to the observed rainfall records by the two-parameter Gamma type storm, known as G2P (García-Bartual and Andrés-Doménech, 2017).

The IDF curve determined for a return period of 15 years has been:

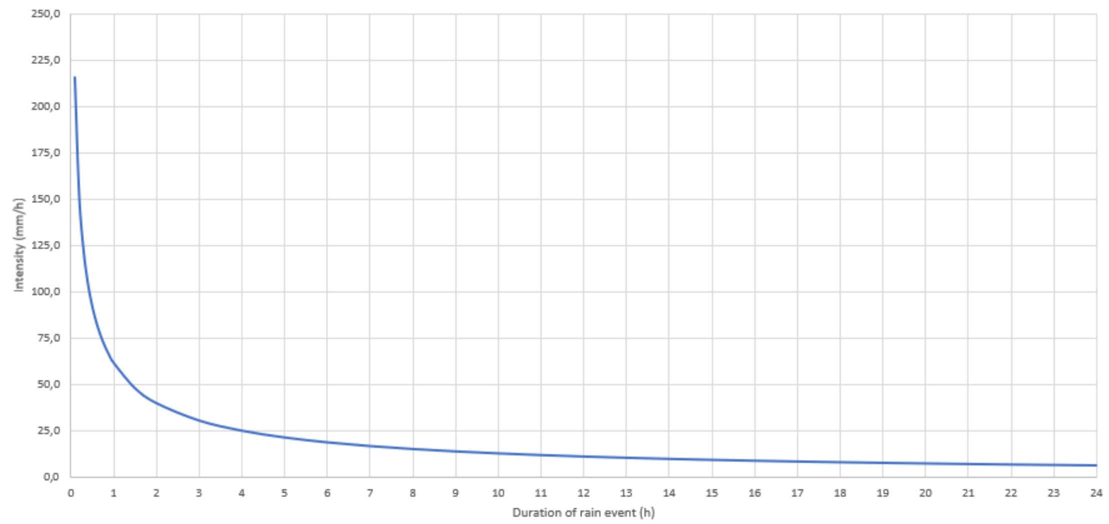

Figure 24. IDF curve for 15 years return period

With respect to design storms from observed precipitation records, three different ones have been determined by the two-parameter Gamma type storm, since the study concludes that for any given return period three different families are associated, all with equivalent magnitudes, but with different intensity, time and volume patterns.

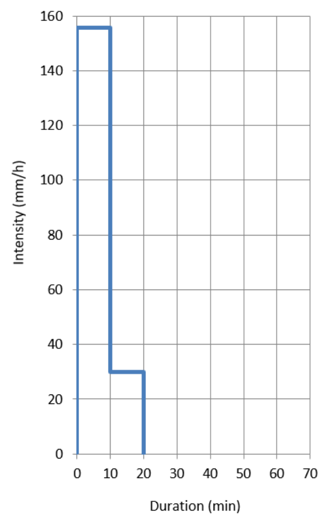

Figure 25. Hyetogram of short-term design storm. T=15 years

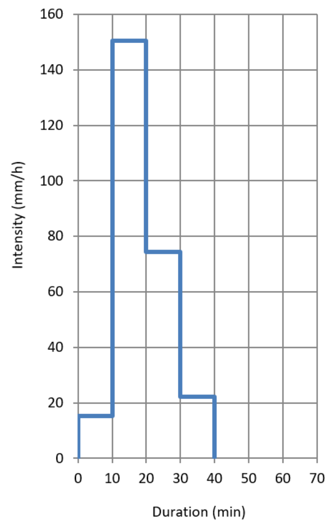

Figure 26. Hyetogram of medium-term design storm. T=15 years

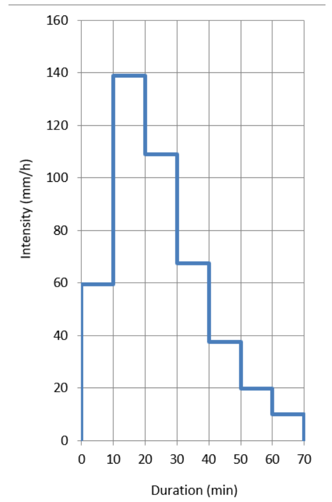

Figure 27. Hyetogram of long-term design storm. T=15 years

The graph below represents the four design storms from the IDF curve and those determined by the G2P method.

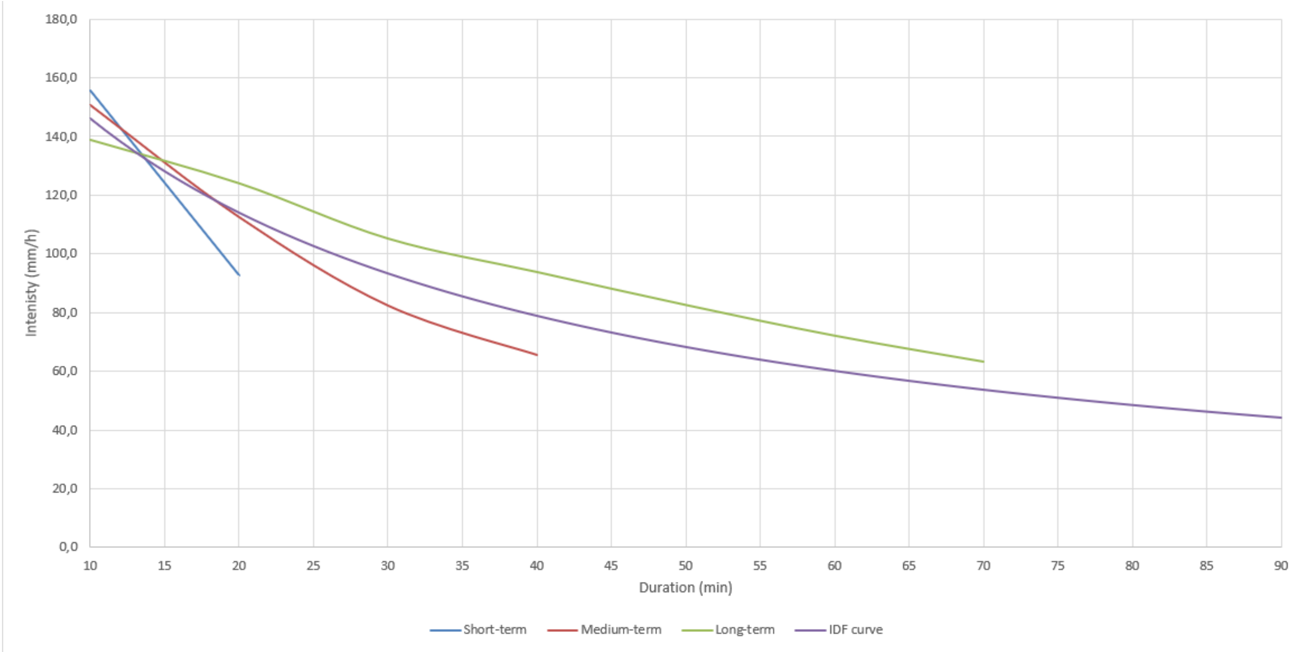

Figure 28. Comparison on design storms T = 15 years

7.8 Natural state

The aim of the Sustainable Drainage Systems in this work is, among others, to return the basins of the study site to the natural state before the urbanisation of the area and impermeabilisation. To do this, the parameters related to the discharge point, the draining catchments and the rainfall-runoff transformation model used for the simulation of the site in its natural state must be known beforehand.

The study area has been divided into three drainage catchments, N1, N2 and N3.

Table 3. Catchments surface

N1	N2	N3	TOTAL
156.712 m2	341.245 m2	696.606 m2	**1.194.563 m2**

At present, rainwater runoff from the catchments diverges naturally into La Saleta ravine without a drainage system, as shown in the following diagram:

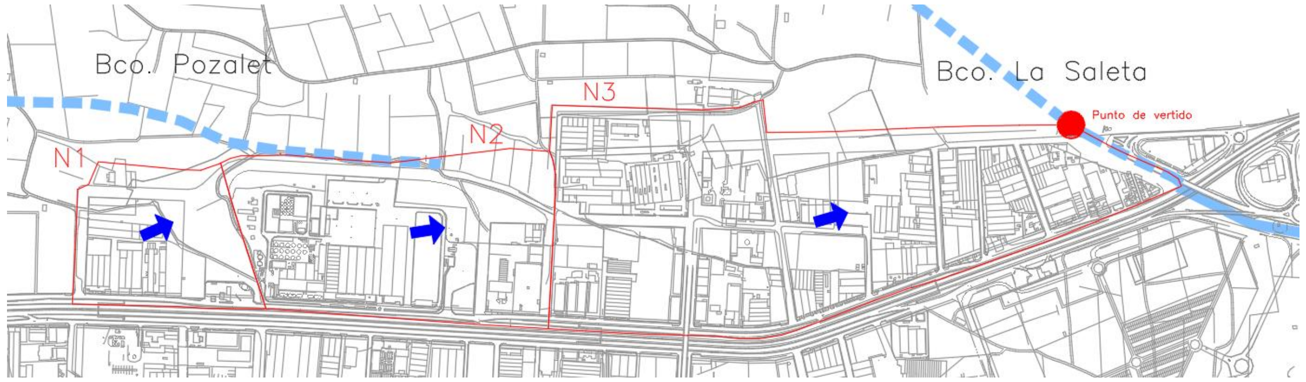

Figure 29. Discharge point and catchments

The estimation of peak flood flows in a natural state is calculated from the rainfall-runoff transformation models. The method used for the rainfall-runoff transformation model is the one included in the regulation 5.2-IC Surface drainage, Orden FOM/298/2016, (Ministerio de Fomento, 2016), proposed in the journal Ingeniería Civil para la Dirección General de Carreteras de España called **Modified Témez Method** (Témez, 1992), whose expression is as follows:

$$Q_T = \frac{I(T, t_c) * C * A * K_t}{3,6}$$

where:

- Q_T (m3/s) is the annual peak flow corresponding to the return period T.
- I(T, t_c) (mm/h) is the rainfall intensity corresponding to the considered return period T, for a storm duration equal to the time of concentration tc of the catchment.
- C (adimensional) is the average runoff coefficient of the catchment.
- A (km2) is the area of the catchment.
- K_t (adimensional) is the coefficient of uniformity in the temporal distribution of rainfall

Applying the modified Témez method, peak flows are determined for each catchment individually and for the whole study area.

Table 4. Peak flows in natural state

	N1	N2	N3	TOTAL
Q (m3/s)	1,77	3,25	4,40	**6,88**

8 SOLUTIONS PROPOSAL

The alternatives proposed to solve the problems of the study and which will serve as input for subsequent verification by means of modelling are as follows:

- The first alternative consists of implementing the entire drainage system in a **conventional system** with a collectors' network.

- The second alternative seeks to improve the response of the system in the previous case by proposing the design of a **stormwater tank** to retain and laminate the water during periods of rainfall and then introduce it into the sewerage network in a controlled way.

- The third alternative, based on the collector system of the first alternative, adds **sustainable drainage systems** such as pervious pavements, rain gardens and infiltration basins to improve the hydrological and hydraulic response of the catchments.

- The fourth and final alternative, to be designed in the case that the third alternative fails to meet the proposed threshold discharge flow, will improve the **sustainable drainage system** response of the third alternative by adding a **stormwater tank**.

Everything mentioned in this section can be found in more detail in *Annex Nº3: Solutions proposal.*

8.1 Design criteria and parameters

The decisive design criteria used for the hydraulic design are:

- The flow system shall preferably be **by gravity**, avoiding as far as possible continuous lifting and pumping systems.

- The operation of the collector shall be in a **uniform stationary regime.**

- The collector regime will be slow regime, limiting the Froude number to $\boldsymbol{Fr < 1}$.. As, in any case, the stable slow regime is assured when $Fr < 0{,}8$, we will try to limit the Froude number to this value.

- The collector will operate at **80% of the full section depth**, as this is the optimum operating capacity.

- The maximum velocity shall be limited to $\boldsymbol{Vmax = 4\ m/s}$ to avoid friction damage.

- The minimum velocity shall be limited to a $\boldsymbol{Vmin = 1{,}2\ m/s}$ to avoid sedimentation of entrained suspended solids and clogging.

- The minimum internal diameter to avoid clogging and to facilitate cleaning is $\boldsymbol{Dmin = 335\ mm}$.

- The frictional energy loss equation is given by the **Manning's equation**.

- The Manning's number of PVC and concrete is $\boldsymbol{n = 0{,}011}$, a conservative value to consider the degradation suffered by the collector.

- **Continuity** will be ensured for **the top line and the energy line**. Therefore, the collector flushing will be done obligatorily through the top line.

- The **energy line** will always be located **below the ground level** and backwaters with a lower energy downstream than the energy upstream will be avoided.

- In the case of the existence of hydraulic backwater, it shall be downstream at a distance of $\boldsymbol{d < 10\emptyset}$

The pipes that will configure the collectors will be of two types depending on their diameter: for diameters of less than 1200 mm, corrugated PVC pipes will be installed. The pipes with diameters greater than 1200 mm will be of rigid PVC, helically shaped with a profile that has a wall structured by means of "T" shaped stiffeners and reinforcement with a galvanised steel profile. The diameters chosen are shown below:

Table 5. Pipe network diameters

DN (mm)	Ø External (mm)	Ø Internal (mm)
400	400	364
500	500	452
630	630	590
800	800	775
1000	1000	970
1200	1200	1103
1300	1300	1268
1400	1400	1368
1500	1500	1468
1600	1600	1568
1700	1700	1668
1800	1800	1768
1900	1900	1868
2000	2000	1968
2100	2100	2068
2200	2200	2168
2300	2300	2268
2400	2400	2368
2500	2500	2468
2600	2600	2568
2700	2700	2668
2800	2800	2768
2900	2900	2868
3000	3000	2968

The design of the sustainable drainage system network will be determined on the basis of volumetric control in order to achieve the quantity and quality objectives. The quantity criteria to be treated in order to achieve the lamination of the flows generated by episodes of rainfall is given by the V_{80} value, while the quality criteria corresponds to being able to store a volume of runoff corresponding to the V_{90} value. In addition, the aim is to maintain the natural drainage pattern of the area, trying to ensure that the evacuation flows to the discharge point do not exceed the flows calculated in a natural state for a return period of 15 years. This objective is much more ambitious and will therefore have larger storage volumes to manage. The following values are determined for the study catchments:

Table 6. V80 value, V90 value and maximum storage volume for T = 15 years

	N1	N2	N3
Storage volume V80 (m3)	1.988	4.609	9.927
Storage volume V90 (m3)	3314	7682	16544
Storage volume of long-term design storm (m3)	9.756	22.617	48.707

In addition, a quality control check has been carried out using the **pollution hazard level index method**, indicating that the mitigation of pollutants by the SUDS should be sufficient to counteract the pollution hazard index of the industrial area.

Table 7. Pollution hazard levels at industrial states. Source: Woods Ballard et al., 2015

Total Suspended Solids	Metals	Hydrocarbons
0,8	0,8	0,9

Common to all scenarios, the sub-catchment division of the study area into private subcatchment, street subcatchment and green area subcatchment is presented.

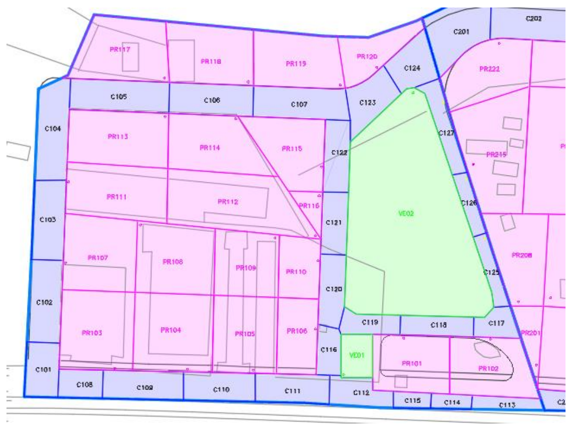

Figure 30. Subcatchment division on catchment N1

Figure 31. Subcatchment division on catchment N2

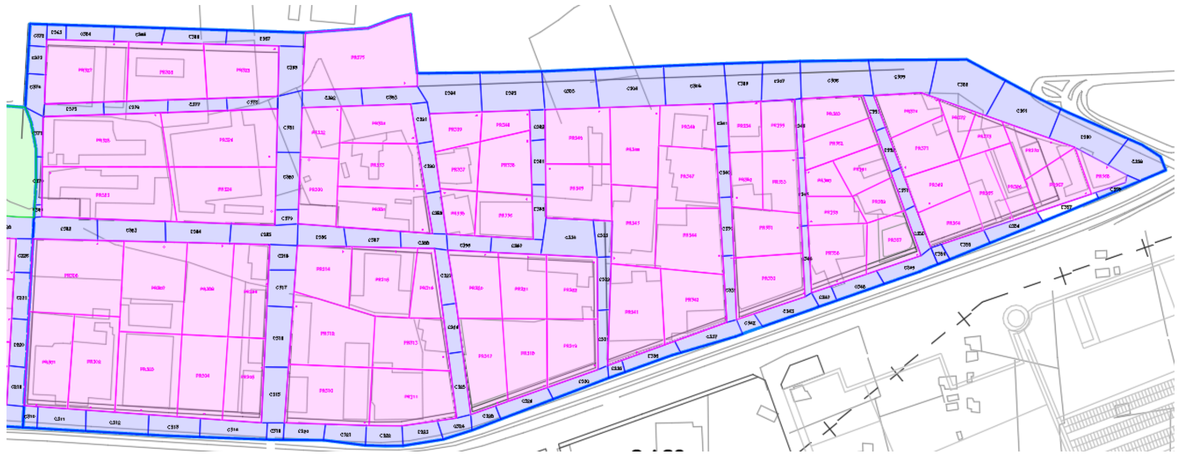

Figure 32. Subcatchment division on catchment N3

From this subcatchment division, a runoff coefficient has been determined for each type, as indicated below:

Table 8. Runoff coefficients for the study area

Surface	Runoff coefficient
Road	0,95
Private catchment	0,75
Green area	0,30
Pervious pavement	0,70
Infiltration basin	0,30
Rain garden	0,30

8.2 Alternative 1. Conventional network solution

The first alternative consists of implementing the entire drainage network in a conventional way with a system of collectors. It cannot be considered as alternative 0, which corresponds to no action, because this alternative is a substantial improvement on the current situation, as there is currently no drainage system in the study area and this alternative would improve the control and conduction of the runoff generated.

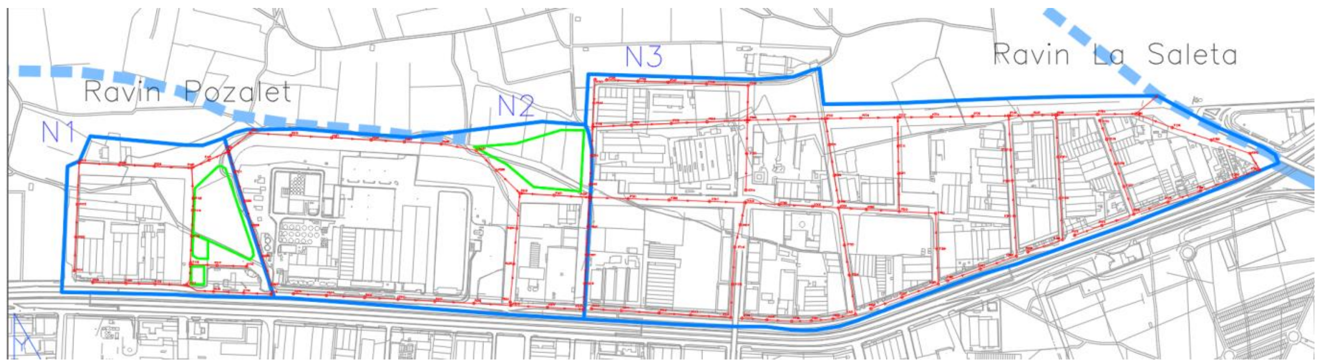

Figure 33. Alternative 1 proposal

8.3 Alternative 2. Stormwater tank solution

Alternative 2 is based on the previous conventional drainage network, which ends in a stormwater tank that allows the lamination and storage of runoff water to reduce the peak flow and control the flow regime towards La Saleta ravine.

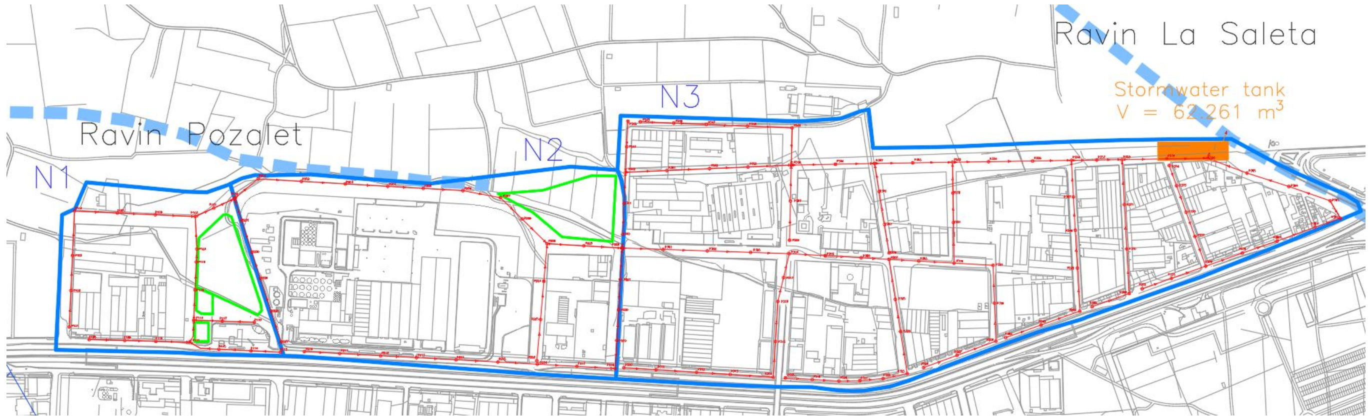

Figure 34. Alternative 2 proposal

The stormwater tank is designed for the volume to be stored and the target peak flow to be discharged into the receiving environment. The tank will be emptied by means of a pumping system and the dimensions of this tank will be as follows:

$$S_{tanque} = \mathbf{6.220\ m2}$$
$$h_{tanque} = \mathbf{10,0\ m}$$

$$V_{tanque} = \mathbf{62.261\ m}$$

8.4 Alternative 3. SuDS solution

Alternative 3 aims to implement a solution that incorporates sustainable drainage systems. By analysing the conditions of the study area, it has been determined that the SuDS best suited to the industrial estate are pervious pavements, rain gardens and infiltration basins.

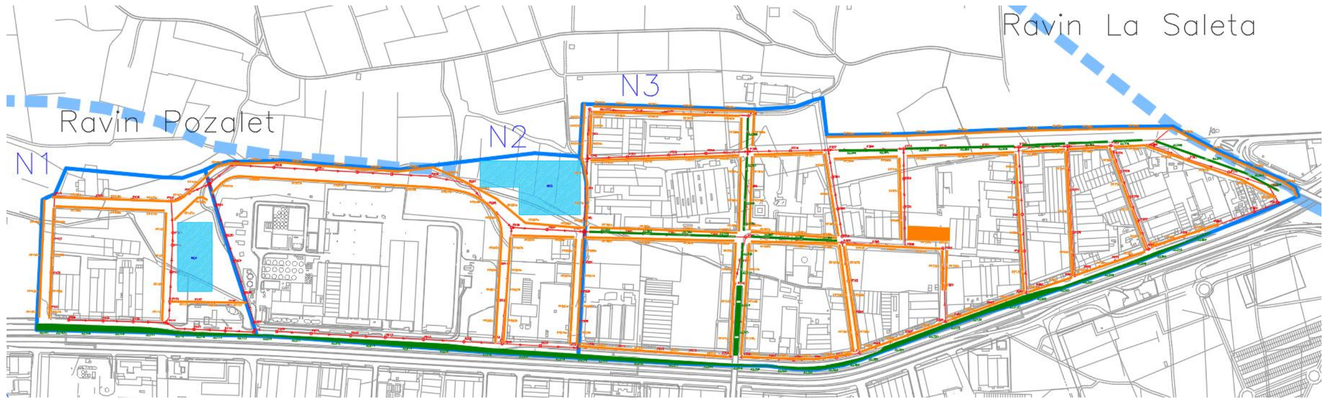

Figure 35. Alternative 3 proposal

The SuDS chain achieves total pollutant mitigation indexes above the pollution risk levels for industrial areas.

Table 9. Total mitigation index at the study area

Total Suspended Solids	Metals	Hydrocarbons
1	0,85	1

Table 10. Pollution risk levels at industrial states. Source: Woods Ballard et al., 2015

Total Suspended Solids	Metals	Hydrocarbons
0,8	0,8	0,9

8.5 Alternative 4. SuDS solution with stormwater tank

Alternative 4 is based on the results of alternative 3. The development of this alternative is carried out in the event that the third alternative does not manage to reach the discharge flow in a natural state, offering a scenario that has a better hydraulic response to the problem.

This alternative is developed based on the sustainable drainage systems of alternative 3 and the design of the stormwater tank of alternative 2 in order to achieve the threshold of discharge flow objective of the work.

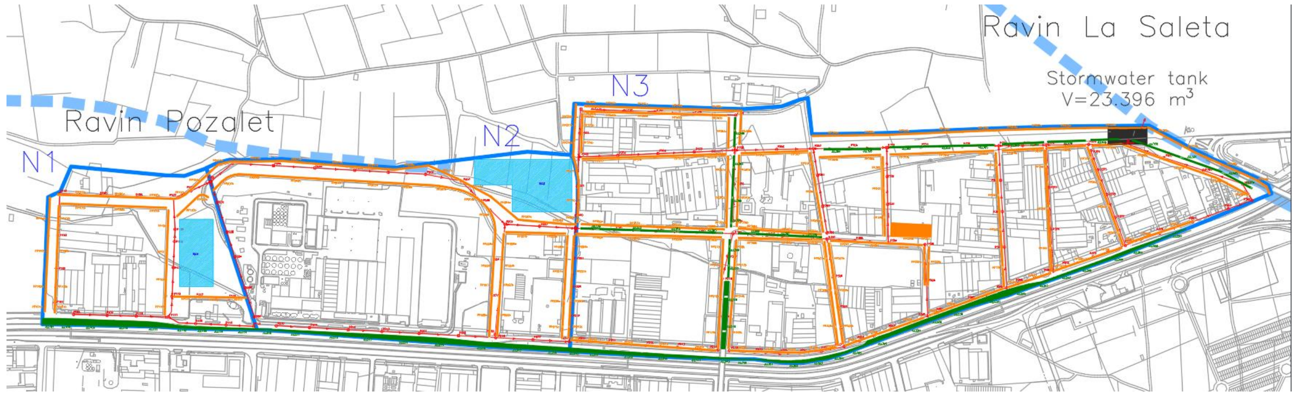

Figure 36. Alternative 4 proposal

The stormwater tank in this case has a much reduced size compared to Alternative 2.

$$S_{tank} = \mathbf{3.655\ m2}$$
$$h_{tank} = \mathbf{6,4\ m}$$

$$V_{tank} = \mathbf{23.396\ m}$$

The emptying system is carried out by means of a pump type CPH 350-360 with an emptying flow rate of 600 l/s to empty the storm tank in less than half a day.

9 MODELLING PROPOSED SOLUTIONS

Mathematical models of a physical system are a simplification of reality that preserves the main characteristics. They are made up of inputs, parameters, equations and state variables.

Mathematical modelling of urban drainage tends to represent the processes of production of runoff from rainfall, transport of runoff on the surface and propagation of runoff in the system of collectors. In the case of our study, the use of a mathematical model is mainly for the design of a new system based on dimensioning calculations and, if necessary, to optimize this design.

Hydraulic design is influenced by:

- **The Saint-Venant equations:** these are the equations governing one-dimensional transitory flow in laminar open flow.

- **The continuity equation:** which establishes the mass balance in each control element.

- **The quantity of motion equation:** which establishes the dynamic equilibrium of the control element.

The program used in this study is the SWMM software, which uses the **explicit finite difference method.** The finite difference method is the most widely used among urban drainage modelling programs and consists of discretising the two-dimensional space (x,t) in a mesh defined from a Δx and a Δt and approximating the partial derivatives of the equations.

As in the previous section, a more in-depth analysis of the programme and the modelling carried out can be found in *Annex Nº3: Solutions proposal.*

9.1 Infiltration model

The modelling of hydrological processes can be done by means of different models. The model used for the study is the empirical SCS (Soil Conservation Service) model of the **curve number (CN)**. Besides the CN model, the SWMM programme offers four other models. The Horton, Modified Horton, Green and Ampt and Modified Green and Ampt models. Although these last four models reproduce much more accurately the processes involved in the production of runoff, they have the significant disadvantage for use in urban drainage models that they depend on too many parameters, which are sometimes difficult to estimate. For this reason, the CN model has been chosen for modelling infiltration.

The CN varies between 0 and 100, the second value corresponding to 100% runoff. The relationship between the CN and the parameters S and P_0 is as follows:

$$S = \frac{25400}{CN} - 254$$

$$P_0 = \frac{5080}{CN} - 50{,}8$$

Where:
- S (adimensional) is the maximum potential difference between rainfall and runoff generated.
- P0 (mm) is the threshold at which runoff is generated.

The values taken for the CN in the study are as follows:

Table 11. Curve number for each zone. Source: Rossman L. A., 2015

Zone	CN – Cond. II
Impermeable – Streets and roadways	98
Edification – Private industrial areas	91
Permeable – Parks and green areas	74

9.2 Hydraulic transport model

The hydraulic transport model used to solve the equations is the **dynamic wave model**, since it solves complete one-dimensional Saint-Venant equations and therefore generates more accurate results. The inconvenience of this model is that it requires smaller time increments, in the order of 1 minute, making the simulation take longer to run. However, the study simulation does not have so many elements, conditions or parameters that this is a limitation.

9.3 Element definitions

The drainage system has been defined in the SWMM program by means of the geometric elements offered by the program. In the modelling of the case study, the the options that will be used are junctions, outfalls, conduits, rain gages, storage units, subcatchments and LID controls.

- **Subcatchments** are hydrological units of the site whose topography and elements of the drainage system convey runoff directly to a discharge point.

- **Junctions** are nodes in the drainage system where the different lines are connected to each other.

- **Outfalls** are terminal nodes of the drainage system used to define the final downstream boundary conditions of the system in the case of using the dynamic wave flow model.

- **Storage units** are nodes of the drainage system with the capacity to store certain volumes of water.

- **Conduits** are pipes or channels through which water moves from one node of the conveyance system to another.

- **Rain Gages** provide the input data for rainfall occurring over one or more of the defined catchments in the study area.

The modelling of sustainable drainage systems, called **LID** (Low Impact Development) in the programme, is carried out according to the following conceptual scheme where, depending on the SuDS typology, the element will be composed of different layers and flows.

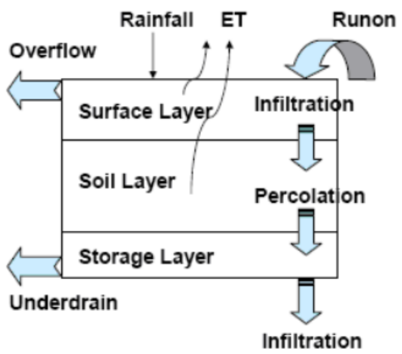

Figure 37. Conceptual diagram of LID representation

9.4 Modelling of alternatives

The different alternatives have been implemented in the SWMM programme on the basis of the previously determined design elements and parameters. The representation of the modelling in the SWMM program can be recognised as follows:

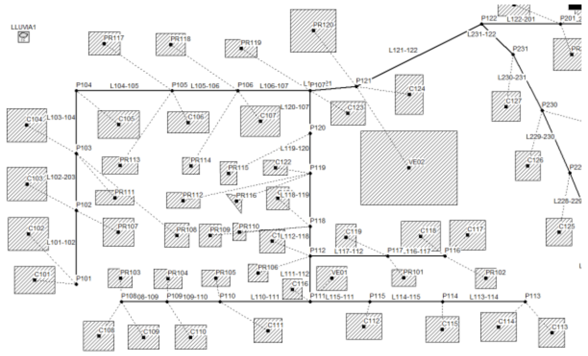

Figure 38. Modelling catchment N1. Alternative 1

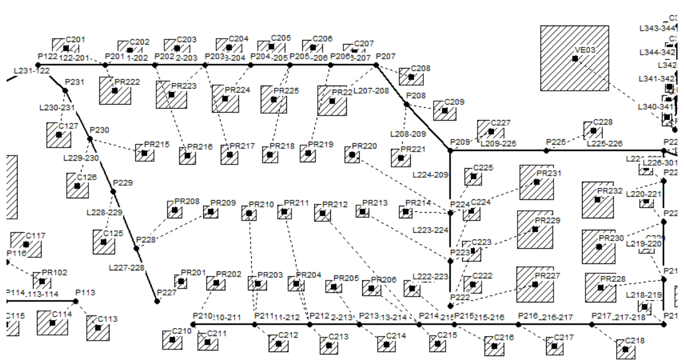

Figure 39. Modelling catchment N2. Alternative 1

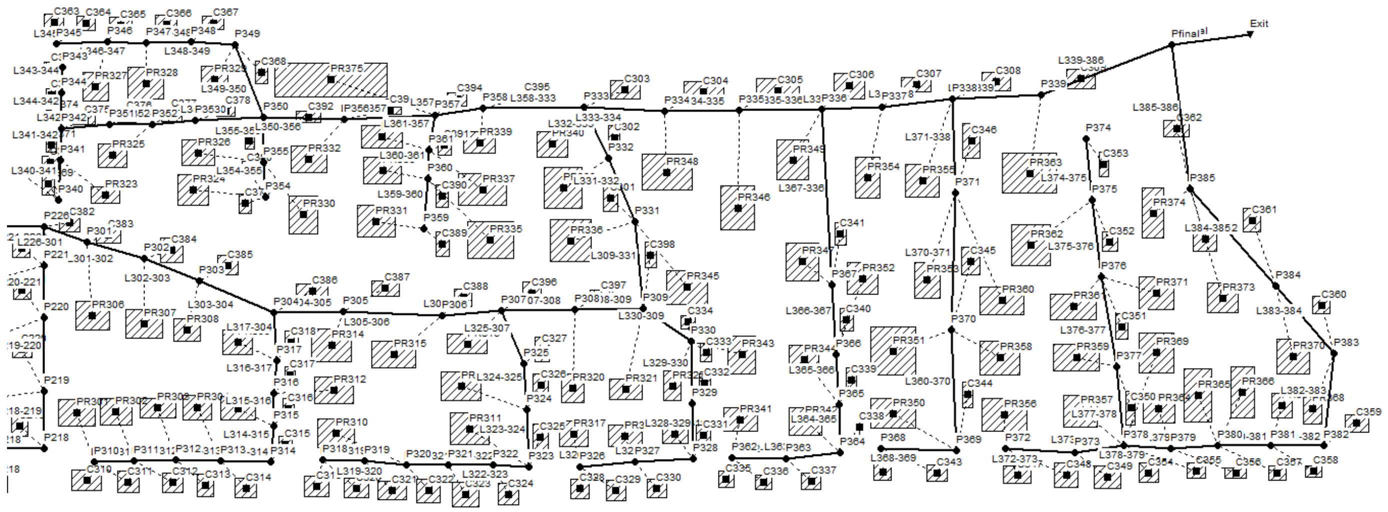

Figure 40. Modelling catchment N3. Alternative 1

The representation of alternative 2 is very similar to the representation of modelling 1, but changing the final connection node to a storage unit.

The modelling representation of alternative 3 keeps the drainage network conduits of modelling 1, but the catchments have been modified to add the SuDS in the subcatchments. Therefore, the areas of the subcatchments have been reduced and another subcatchment representing the SuDS has been added. The N1 and N2 catchments have a similar connection logic system, while the N3 catchment is slightly more complex.

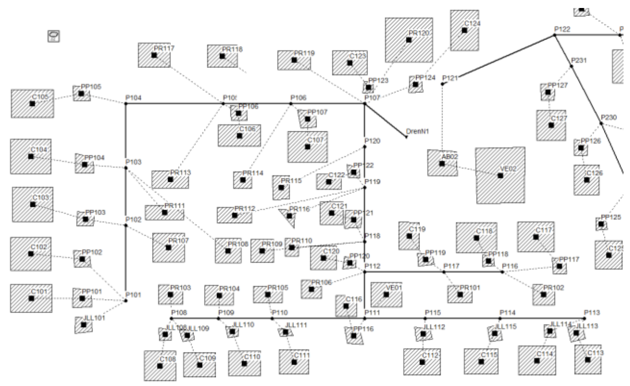

Figure 41. Modelling catchment N1. Alternative 3

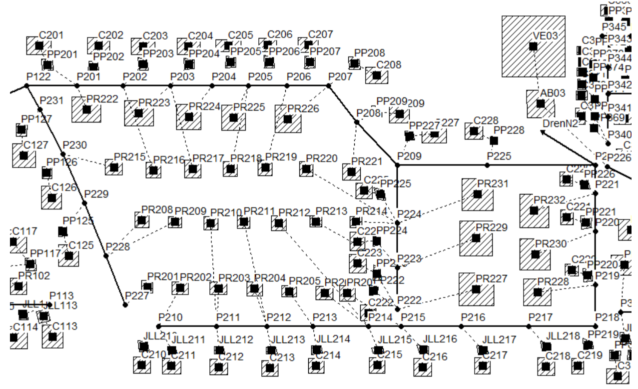

Figure 42. Modelling catchment N1. Alternative 3

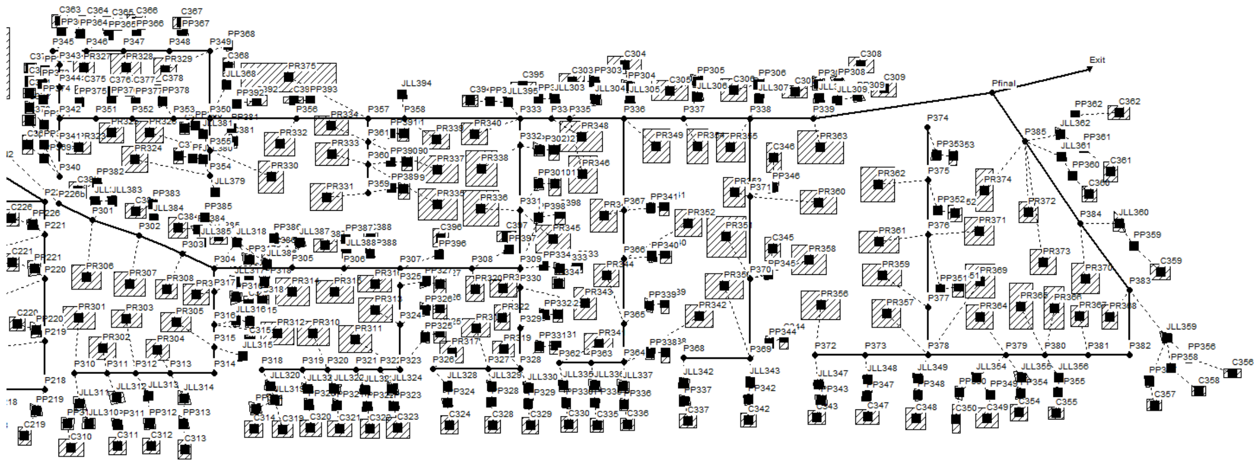

Figure 43. Modelling catchment N1. Alternative 3

The representation of alternative 4 is based on modelling 3 with the addition of a storage unit in the final section with enough volume to be able to manage the runoff water so that the final discharge is equal to that of the natural state.

The most remarkable results of all the alternatives, corresponding to the peak flow and volume discharged, have been compiled in order to be able to make a summary comparison.

Table 12. Comparison on alternatives' peak flow

Design storm	Peak flow (m3/s)			
	Alt 1	Alt 2	Alt 3	Alt 4
Short-term	24,49	0,00	12,69	0,00
Medium-term	29,71	0,00	14,14	0,00
Long-term	33,29	6,88	15,91	6,88

Table 13. Comparison on alternatives' peak flow reduction

Design storm	Peak flow reduction			
	Alt 1	Alt 2	Alt 3	Alt 4
Short-term	-	100%	48,2%	100%
Medium-term	-	100%	52,4%	100%
Long-term	-	79,3%	52,2%	79,3%

Table 14. Comparison on alternatives' discharge volume

Design storm	Discharge volume (m3)			
	Alt 1	Alt 2	Alt 3	Alt 4
Short-term	25.945	0	13.101	0
Medium-term	40.208	0	20.667	0
Long-term	74.809	18.222	39.584	17.868

10 COMPARATIVE MULTI-CRITERIAL ANALYSIS

The comparative analysis of the proposals in order to find the optimum solution was carried out using the E2Stormed software, which functions as a Decision Support Tool that helps in the decision-making process by analysing the advantages and disadvantages of drainage systems for stormwater management, taking into account financial, hydraulic, energy, environmental and social criteria.

The multi-criteria analysis in this section can be found explained in detail and in greater depth on the interpretation of the results in *Annex Nº4: Comparative multi-criterial analysis.*

In order to carry out the analysis, the information is first entered for each proposal with respect to the following aspects:

- Water supply
- Stormwater runoff
- Conveyance and treatment
- Water quality
- Flood protection
- Building insulation
- Ecosystem services

Subsequently, all the infrastructures of each alternative are included, including the cost, the energy consumed and the emission of pollutants for the construction and maintenance process.

In this way, the result is obtained for each alternative:

Table 15. Results of cost, energy and emissions for each alternative

Infrastructure	CONSTRUCTION			MAINTENANCE		
	Cost (€)	Energy consumed (kWh)	Emissions (kgCO2e)	Cost (€/year)	Energy consumed (kWh/year)	Emissions (kgCO2e/year)
Alternative 1	20.203.700	57.042.850	17.962.588	295.348	6.055	1.594
Alternative 2	51.334.200	109.921.495	34.712.042	388.740	6.063	1.596
Alternative 3	22.252.072	53.657.674	16.895.614	550.329	5.380	1.417
Alternative 4	33.950.072	73.527.662	23.189.606	585.423	5.388	1.419

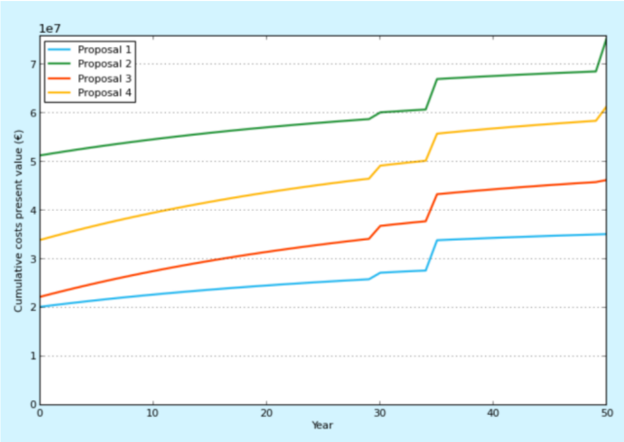

Figure 44. Cumulative total costs on present value during the lifespan

After the analysis of the results, it is determined that the most economical solution is proposal 1, as expected, since the rest of the proposals are a modification of proposal 1 and therefore their construction and maintenance cost would be higher. Considering that the construction and maintenance costs are of a greater magnitude than the other costs, the difference in construction and maintenance costs between the alternatives plays a crucial role in the economics of the solutions.

E2stormed also provides the comparative results of energy consumption throughout the project. The results identify proposals 2 and 4 as the most energy consuming proposals. This is mainly due to the implementation of a pumping system to lift the water from the stormwater tank to the level where the sewage network is located. Proposal 2 consumes more energy because the tank is larger and must draw more water from a greater depth.

The comparative cumulative emissions of each proposal are presented in the graph below. The emissions of the proposals are presented in accordance with those of energy consumption, as energy consumption is one of the largest emitters of CO2 equivalent. In the proposed scenarios, in no case do emissions decrease over time.

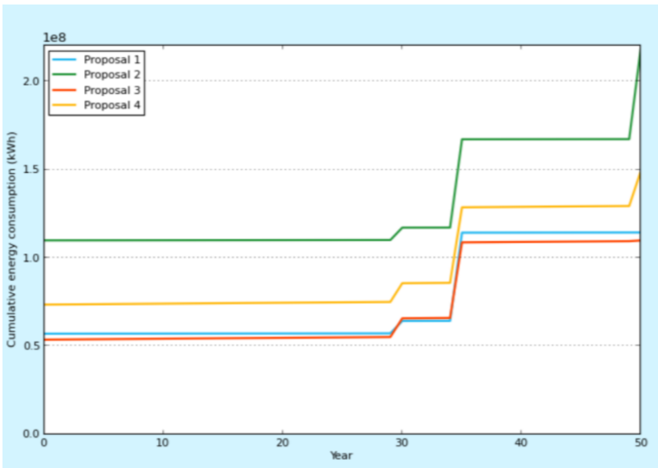

Figure 45. Cumulative energy consumption during lifespan

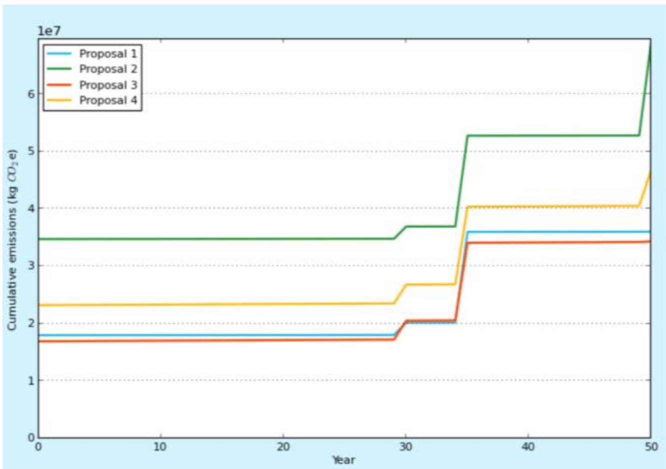

Figure 46. Cumulative emissions during the lifespan

Afterwards, the criteria that will determine the optimal solution must be chosen and a weight must be added to each one of them, in our case, six criteria have been considered as the determining factors in the choice of alternatives.

Table 16. Chosen weight for criteria

Criterion		Weight
Net cost of stormwater management	C1	40%
Peak outflow rate	C2	35%
Global outflow water quality	C3	7%
Evaluation of ecosystem services	C4	7%
Net energy consumed by stormwater management	C5	5,5%
Net emissions of stormwater management	C6	5,5%

In this way, the results are obtained for each alternative in the form of a diagram representing each criteria:

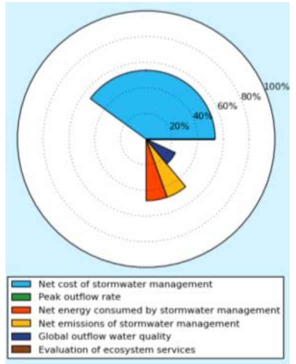

Figure 47. Diagram of result multi-criteria analysis. Alternative 1

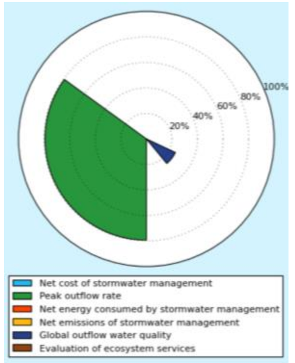

Figure 48. Diagram of result multi-criteria analysis. Alternative 2

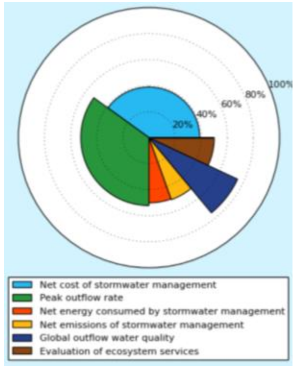

Figure 49. Diagram of result multi-criteria analysis. Alternative 3

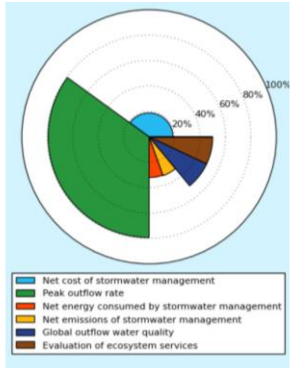

Figure 50. Diagram of result multi-criteria analysis. Alternative 4

Finally, from these results we obtain the overall score for each alternative by multiplying the above results by the weight of each criteria. This allows us to compare all scenarios.

Table 17. Overall score for multi-criteria analyse

Criteria	Alternative 1	Alternative 2	Alternative 3	Alternative 4
Net cost of stormwater management (%)	21,44	0,00	15,55	7,49
Peak outflow rate (%)	0,00	27,77	18,27	27,77
Global outflow water quality (%)	1,75	1,75	5,25	3,5
Evaluation of ecosystem services (%)	0,00	0,00	3,5	3,5
Net energy consumed by stormwater management (%)	2,64	0,00	2,76	1,76
Net emissions of stormwater management (%)	2,65	0,00	2,78	1,79
TOTAL (%)	**28,49**	**29,52**	**48,12**	**45,81**

Its graphical representation is made by means of a histogram that identifies the results of each criterion applied to each alternative, which is presented in the following image:

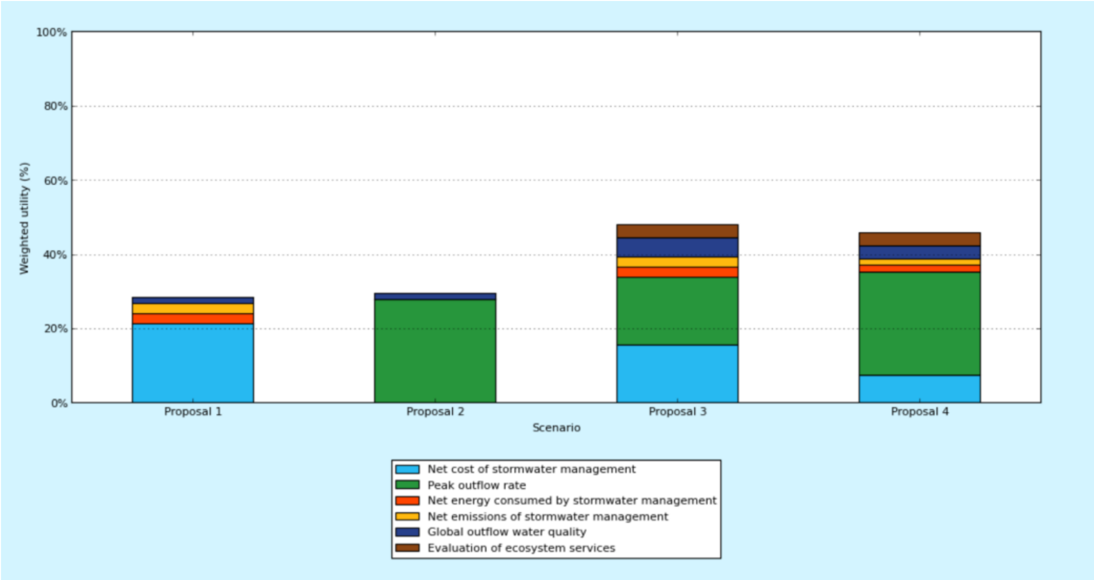

Figure 51. Histogram of global results of multri-criteria analysis

After the multi-criteria analysis, it is concluded that the **optimum solution is alternative 3**, where sustainable drainage systems are implemented but it does not include a storm tank and therefore does not reach the target peak flow in the natural state. However, alternative 4 is located very close to it. Analysing in detail the reasons why alternative 3 ranks higher than alternative 4, it has been concluded that alternative 3 ranks first due to the energy, emission and environmental benefits of its drainage system, although in the combined aspects of hydraulics and finance it is inferior to alternative 4.

Table 18. Close analysis between alternative 3 and 4

Criteria	Proposal 3	Proposal 4
Financial and hydraulic criteria (%) C1 + C2	33.82	35.26
Environmental and energetic criteria (%) C3 + C4 + C5 + C6	14.3	10.55
TOTAL (%)	**48,12**	**45,81**

This indicates that, for our analysis, the benefit of constructing a storm tank in terms of discharge flow makes it the best option even taking into account its costs. However, the added implementation of the tank and its pumping system, with its consequences in terms of energy and environmental criteria, means that this alternative falls into second position. Finally, it is interpreted that the proximity of proposals 3 and 4 suggests that the prioritisation of the alternatives is considerably sensitive to the weights given to the criteria.

11 DEVELOPMENT OF THE OPTIMAL SOLUTION

Identifying alternative 3 as the optimal one, the adopted solution has been developed, which can be found in detail in *Annex Nº5: Development of the optimal solution*. The most outstanding characteristics of the sections of the sustainable drainage systems to be implemented in the study area are presented below.

11.1 Pervious pavements

The pervious pavement section shall be composed of the following layers:

- Superficial porous concrete layer
- Sub-superficial gravel layer 4/8
- Geotextile
- Sub-base gravel layer 20/40
- Drain
- Geomembrane

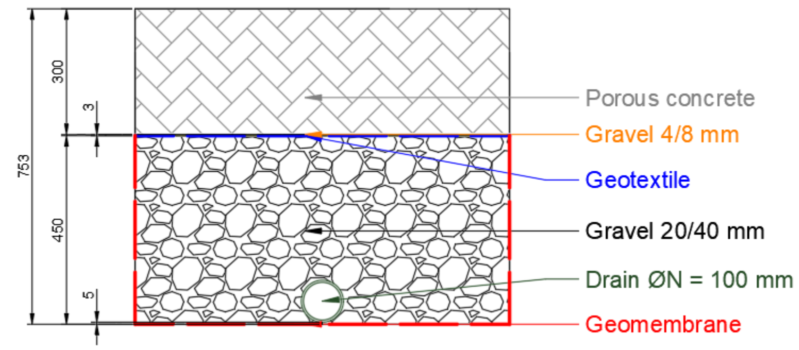

Figure 52. Cross-section on pervious pavement

The porous concrete surface layer shall be the first top layer of the section that will be in contact with the vehicles. This layer shall have the following characteristics:

- The thickness of the layer shall be 300 mm.
- The void ratio for the porous mix shall correspond to 25%.
- Its stiffness shall be between 25 and 45 GPa, with a common value of 38 GPa.
- The specific weight of the porous concrete shall be between 1700 and 2000 kg/m3.
- A particle size without fines or at least with a low content shall be required. Identifying a minimum aggregate size of 5mm. The maximum aggregate size shall be 20 mm.
- The binder used shall be hydraulic cement, with a content of 267-326 kg/m3 and a water-cement ratio of 0.26-0.38. The cement shall be modified with silica fume admixtures, superplasticiser and organic polymers to move from average compressive strengths of 20 MPa to ensure a compressive strength more than 45 MPa.
- The minimum flexural tensile strength shall be 2.5 MPa.
- The permeability of the porous concrete shall be around 250 mm/h.

A sub-layer of gravel shall be placed under the porous concrete surface layer with a thickness of 30 mm and a minimum aggregate size of 4 mm and a maximum of 8 mm, with the aggregate size increasing with depth.

A non-woven geotextile sheet will be placed between the surface layer and the storage layer, which will act as a filter and separation to prevent the transfer of fines and the loss of support at the base. This sheet will also retain most of the pollutants carried by the filtered water. Its characteristics will be:

- Mass per unit surface area between 125 and 160 gr/m2.
- Vertical permeability between 100 and 130 mm/s.

The storage layer of gravel shall consist of the following specifications:

- The thickness shall be 450 mm.
- The aggregates shall be artificial and from quarries.
- The granulometry shall be as uniform as possible, trying to eliminate fines smaller than 2 mm.
- The voids index shall be 50%.
- The Los Angeles coefficient shall be less than 30.
- The sand equivalent shall be higher than 40.
- The particle size shall be between 20 and 40 mm.

Finally, a layer of geomembrane shall be laid to impermeabilize the entire structure and thus prevent infiltration. This layer must ensure a total sealing of the esplanade that will allow for a waterproof structure that will function as a storage tank for the filtered water.

The outlet of the stored water will be by means of a PVC drainage pipe buried in the gravels at 5 mm from the base. The nominal diameter of the pipe will be 100 mm and it will be lined with a geotextile to avoid obstructions in its grooves.

11.2 Rain garden

The rain gardens have two types of sections. One with geomembrane and drainage and one without either, thus allowing infiltration.

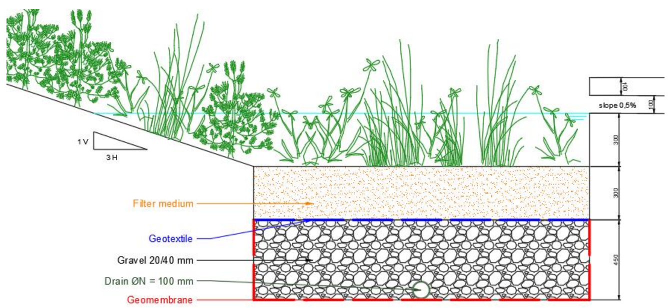

Figure 53. Cross-section on rain gardens without infiltration

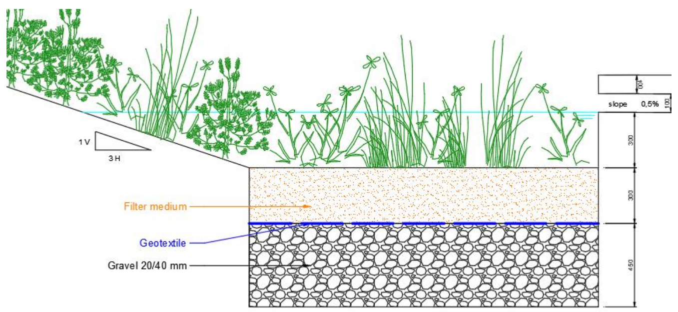

Figure 54. Cross-section on rain gardens with infiltration

The cross-sections shall consist of the following layers:

- Free surface
- Filter medium
- Geotextile
- Sub-base gravel layer 20/40
- Geomembrane (if no infiltration)
- Drain (if no infiltration)

The free surface area available in the rain gardens allows a water height of up to 300mm to be stored as a storage tank. For events over the design height, a relief structure, normally a box spillway, with a height of 100mm is included to safely direct runoff downstream, with a slope of 0.5%. A buffer shall also be provided between the top of the spillway and the finished elevation of the adjacent 100 mm surface. The free surface shall be covered with native vegetation, with a minimum density of 6 pcs/m2 and a minimum vegetation volume fraction of 0.1/m3.

The filter medium has the following characteristics:

- The thickness shall be 300 mm.
- The permeability shall be 250 mm/h
- The porosity will be 0.5
- The organic matter content will be between 3-5%.
- The pH will be between 5.5 and 8.5
- Electrical conductivity will be less than 3300 microS/cm

A non-woven geotextile sheet is placed between the filter medium and the storage layer to ensure that soil particles do not pass from one layer to the other. This sheet has the same functions as in pervious pavements and the same characteristics:

- Mass per unit area between 125 and 160 gr/m2.
- Vertical permeability between 100 and 130 mm/s.

The gravel sub-base shall be more permeable than the filter medium.

- The thickness shall be 450 mm
- Aggregate size shall be between 20 and 40 mm.
- The void ratio shall be 50%.
- The aggregates shall be artificial and from quarries.

In the event that the rain garden does not allow infiltration, i.e. when they are at the head of the pollutant mitigation chain, a geomembrane sheet will be provided to waterproof the structure and the stored water will be drained by means of a PVC drainage pipe buried in the gravel 5 mm from the base. The nominal diameter of the pipe shall be 100 mm and it shall be lined with a geotextile to avoid obstructions in its grooves.

11.3 Infiltration basins

Infiltration basins have a similar cross-section to rain gardens that allow infiltration, except that these basins have a free surface area that allows for greater water infiltration and do not have a bioretention function, as SuDS bioretention elements cannot be larger than 0.8 ha in size.

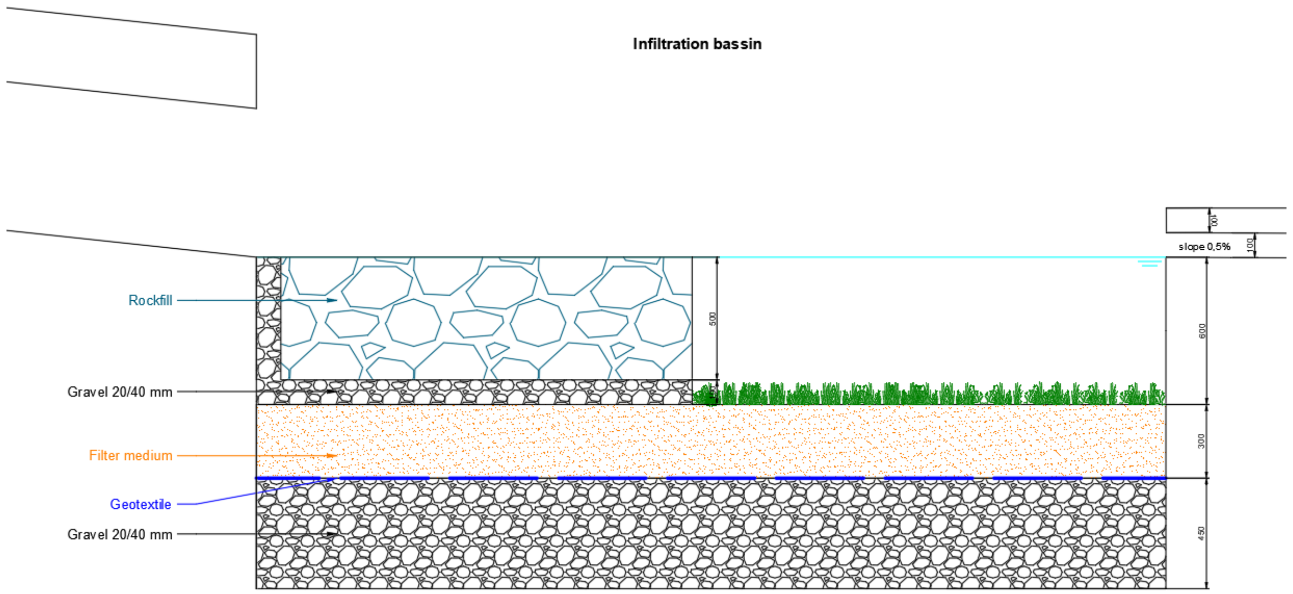

Figure 55. Cross-section of infiltration basin

The cross-section shall consist of the following layers:

- Free surface
- Rockfill layer
- Subbase gravel 4/8 layer under rockfill
- Filter medium
- Geotextile
- Subbase gravel 20/40 layer

The free surface area available in the infiltration areas allows for the storage of up to 600 mm of water as a reservoir, which is completely emptied within 48 hours, both at the base and at the sides. For this reason, the inlet infrastructure will be placed with the floor level at this height. This infrastructure is a pipe whose diameter varies depending on whether we are in the infiltration basin of basin N1 or N2. As in the rain gardens, a relief structure is included, normally a box-type spillway, with a height of 100 mm to direct the runoff downstream in safe conditions, with a slope of 0.5% for events exceeding the design slope. Moreover, a certain buffer between the upper level of the spillway and the finished level of the adjacent surface of 100 mm will also be proposed. In addition, the free surface shall be covered with native vegetation, with a minimum density of 16 pcs/m2.

The entrance shall have an energy dissipating element composed of a layer of rockfill with an average diameter of 250 mm. It shall be trapezoidal in shape and 50 mm thick. A layer of 100 mm thick gravel 20/40 mm shall be placed under the rockfill layer.

The filter medium has the following characteristics:

- The thickness shall be 300 mm.
- The permeability shall be 250 mm/h
- The porosity will be 0.5
- The organic matter content will be between 3-5%.
- The pH will be between 5.5 and 8.5
- Electrical conductivity will be less than 3300 microS/cm

A non-woven geotextile sheet is placed between the filter medium and the storage layer to ensure that soil particles do not pass from one layer to the other. This sheet has the same functions as in pervious pavements and the same characteristics:

- Mass per unit area between 125 and 160 gr/m2.
- Vertical permeability between 100 and 130 mm/s.

The gravel sub-base shall be more permeable than the filter medium.

- The thickness shall be 450 mm
- Aggregate size shall be between 20 and 40 mm.
- The void ratio shall be 50%.
- The aggregates shall be artificial and from quarries.

12 MONITORING AND MAINTENANCE PROGRAM

A monitoring and maintenance plan has been determined for the optimal solution. The monitoring plan is developed to know the state of the infrastructure and to follow up on it while the maintenance plan deals with performing inspections to determine the level of efficiency and plan maintenance needs, performing operations and maintenance of the drainage system, managing the landscape and managing waste associated with contaminated soils or material as well as waste generated by maintenance. These plans are further developed in *Annex Nº6: Monitoring and maintenance program*

The monitoring plan will seek to track water quality and quantity at three different monitoring points:

Figure 56. Monitoring points

Quantity will be monitored using ultrasonic flowmeters and hydrostatic pressure sensors. Quality will be assessed by the following parameters:

Table 19.Quality analysis parameters

Parameter	Analysis method	Range
Total Suspended Solids	UNE – EN 872	-
Biological Oxygen Demand after 5 days	Respirometry	0 – 80 mg O2/L
Chemical Oxygen Demand	ISO 15705	4 - 40 mg O2/L 10 - 150 mg O2/L 25 - 1500 mg O2/L
Turbidity	Turbidimetry	0,02 – 800 NTU
Total Nitrogen	ISO 11905 – 1	0,20 – 20 mg O2/L
Total Phosphorus	ISO 6878/1	0,01 – 5 mg P/L

With regard to the maintenance plan, the following tasks have been determined, indicating their typology, frequency and the component they affect for each sustainable drainage system.

Table 20. Pervious pavement maintenance proposal

Task	Type	Frequency	Element
Initial inspection after construction	Inspection	Monthly for the first three months	Pavement surface
Routine inspection for clogged areas or unwanted vegetation growth	Inspection	Six-monthly and after heavy rainfall	Pavement surface
Permeability testing to determine the need for corrective action	Inspection	Annually for the first three years	Pavement surface
Dry sweeping or standard vacuuming	Periodic	Six-monthly (spring and autumn)	Pavement surface
Remove unwanted vegetation on pavement and adjacent areas	Occasiona l	Annually or as needed	Pavement surface
Correction of vegetation or soil levels on adjacent surfaces that have risen up to 50 mm above pavement level	Corrective	Every 5 years or as needed	Pavement surface
Repairing any depressions, cracks or broken pavers that compromise the structural capacity of the pavement or present a risk to users and replacing joint material	Corrective	Every 5 years or as needed	Pavement surface
Rehabilitation of the surface and top of the sub-base by deep vacuuming, if permeability has been significantly reduced by clogging	Corrective	Every 15 years	Pavement surface
Reconstruction at the end of lifespan	Corrective	At end of lifespan (25 years)	Pavement surface

Table 21. Rain gardens maintenance proposal

Task	Type	Frequency	Element
Remove leaves, trash, sediment and unwanted weeds from the infiltration surface	Periodic	Monthly	Surface, spillway and inlet
Remove sediment accumulated at the inlet and sediment that gets trapped in the spillway	Periodic	Annually or as needed	Surface, spillway and inlet
Irrigation to maintain good vegetation density	Periodic	Monthly	Vegetation
Replant sparsely vegetated areas	Corrective	When necessary	Vegetation
Regular checking for sediment accumulation, ponding, vegetation damage and erosion	Inspection	Six-monthly	Surface, spillway, inlet and vegetation
Technical inspection of inlet structures and infiltration surface (if applicable)	Inspection	Six-monthly and after heavy rains	Surface
Evaluation inspection of emptying time	Inspection	Six-monthly and after heavy rains	Surface
Filling of eroded areas and improvement of erosion protection if necessary	Corrective	When necessary	Surface, spillway, inlet and vegetation
Testing of permeability of the filter medium to determine the need for corrective action	Inspection	Biennially	Filter medium
Remove silt accumulations, scarify surface and backfill with substrate	Occasional	Every 3 years or as needed	Filter medium
Technical inspection for evidence of animal evidence in the interior	Inspection	Annually	Filter medium
Perform permeability test to determine need for corrective action	Inspection	Annually, for the first three years	Surface
Reconstruction at the end of lifespan	Corrective	At end of lifespan (30 years	Surface, vegetation, inlet, spillway, filter medium, gravels, drainage

Table 22. Infiltration basins maintenance proposal

Task	Type	Frequency	Element
Remove leaves, litter and unwanted weeds from the infiltration surface	Periodic	Monthly	Surface, spillway and inlet
Remove accumulated sediments at the inlet and those trapped in the spillway	Periodic	Annually or as needed	Surface, spillway and inlet
Technical inspection of inlet and outlet structures and spillway for blockages	Inspection	Six-monthly and after heavy rains	Surface
Inspection of the infiltration surface	Inspection	Six-monthly and after heavy rains	Surface
Irrigation to maintain good vegetation density	Periodic	Monthly	Vegetation
Cutting vegetation and removing weeds	Periodic	Monthly	Vegetation
Replant poorly vegetated areas	Corrective	When necessary	Vegetation
Inspection of the basin for puddles showing evidence of clogged areas	Inspection	Annually	Surface
Regular check for sediment accumulation and erosion	Inspection	Annually	Surface, spillway, inlet and vegetation
Verification that no settling is happening	Inspection	Annually	Surface
Inspection to evaluate emptying time	Inspection	Six-monthly and after heavy rains	Surface
Fill in eroded areas and improve erosion protection if necessary	Corrective	When necessary	Surface, spillway, inlet and vegetation
Perform permeability test to determine need for corrective action	Inspection	Biennially	Surface, gravel
Inspect slopes and verify topography	Occasional	Biennially	Surface
Level basin base and reinstate design levels	Occasional	When necessary	Surface
Reconstruction at the end of lifespan	Corrective	At end of lifespan (30 years	Surface, vegetation, inlet, spillway, filter medium, gravel and drainage

13 ECONOMIC ASSESSMENT

The economic assessment is included in *Document Nº3: Economic assessment*, where it is determined that the budget of the project amounts to TWENTY-FOUR MILLION FOUR HUNDRED AND FIFTY-NINE THOUSAND FOUR HUNDRED AND THIRTY EUROS AND TWENTY SEVEN CENTS.

14 SUSTAINABLE DEVELOPMENT GOALS. UN 2030 AGENDA

The United Nations adopted in 2015 the 2030 Agenda for Sustainable Development, publishing the 17 Sustainable Development Goals that will promote economic growth, commitment to social needs and protection of the environment.

Figure 57. Sustainable development goals. Source: UN

The Sustainable Development Goals identified in the 2030 Agenda are:

- Goal 1. End poverty in all its forms everywhere
- Goal 2. End hunger, achieve food security and improved nutrition and promote sustainable agriculture.
- Goal 3. Ensure healthy lives and promote well-being for all at all ages.
- Goal 4. Ensure inclusive and equitable quality education and promote lifelong learning opportunities for all.
- Goal 5. Achieve gender equality and empower all women and girls.
- Goal 6. Ensure availability and sustainable management of water and sanitation for all.
- Goal 7. Ensure access to affordable, reliable, sustainable and modern energy for all.
- Goal 8. Promote sustained, inclusive and sustainable economic growth, full and productive employment and decent work for all.
- Goal 9. Build resilient infrastructure, promote inclusive and sustainable industrialization and foster innovation.
- Goal 10. Reduce inequality within and among countries.
- Goal 11. Make cities and human settlements inclusive, safe, resilient and sustainable.
- Goal 12. Ensure sustainable consumption and production patterns.
- Goal 13. Take urgent action to combat climate change and its impacts.
- Goal 14. Conserve and sustainably use the oceans, seas and marine resources for sustainable development.
- Goal 15. Protect, restore and promote sustainable use of terrestrial ecosystems, sustainably manage forests, combat desertification, and halt and reverse land degradation and halt biodiversity loss.
- Goal 16. Promote peaceful and inclusive societies for sustainable development, provide access to justice for all and build effective, accountable and inclusive institutions at all levels.
- Goal 17. Strengthen the means of implementation and revitalize the global partnership for sustainable development.

This final master thesis, in a context in which the commitment of the scientific community to improving society within the framework of the Sustainable Development Goals is sought, has pursued the achievement of certain objectives mentioned above. In particular, the objectives aimed at in this work have been:

- **Goal 6.** Ensure availability and sustainable management of water and sanitation for all
- **Goal 9.** Build resilient infrastructure, promote inclusive and sustainable industrialization and foster innovation.
- **Goal 11.** Make cities and human settlements inclusive, safe, resilient and sustainable
- **Goal 13.** Take urgent action to combat climate change and its impacts.
- **Goal 15.** Protect, restore and promote sustainable use of terrestrial ecosystems, sustainably manage forests, combat desertification, and halt and reverse land degradation and halt biodiversity loss.

The contributions of the work to the achievement of each of the objectives are described below, mentioning the specific targets that have been pursued:

Goal 6. Ensure availability and sustainable management of water and sanitation for all

One of the main objectives to which the work contributes is sustainable water management through sustainable drainage systems. Specifically, the targets indicated by this objective that are being pursued are 6.3, 6.5 and 6.b.

6.3 By 2030, improve water quality by reducing pollution, eliminating dumping and minimizing release of hazardous chemicals and materials, halving the proportion of untreated wastewater and substantially increasing recycling and safe reuse globally.

6.5 By 2030, implement integrated water resources management at all levels, including through transboundary cooperation as appropriate.

6.B Support and strengthen the participation of local communities in improving water and sanitation management

9 INDUSTRIES, INNOVATION AND INFRASTRUCTURE

Goal 9. Build resilient infrastructure, promote inclusive and sustainable industrialization and foster innovation

With a lower relation compared to the previous objective, the development and promotion of sustainable infrastructures such as sustainable drainage systems is pursued in this objective. In addition, industrial areas are mentioned in a certain way, although it does not go into detail on the development of sustainable industrial activities, it is about the development of sustainable measures in the field of industrialisation. The target mentioned in this work is 9.1.

9.1 Develop quality, reliable, sustainable and resilient infrastructure, including regional and transborder infrastructure, to support economic development and human well-being, with a focus on affordable and equitable access for all.

11 SUSTAINABLE CITIES AND COMMUNITIES

Goal 11. Make cities and human settlements inclusive, safe, resilient and sustainable

This objective is pursued on different fronts, the first one being related to floods and the consequences caused by natural disasters, but it also deals with the efficient use of resources and the implementation of related policies. The targets of this objective pursued in the work are 11.5 and 11.b.

11.5 By 2030, significantly reduce the number of deaths and the number of people affected and substantially decrease the direct economic losses relative to global gross domestic product caused by disasters, including water-related disasters, with a focus on protecting the poor and people in vulnerable situations

11.B By 2020, substantially increase the number of cities and human settlements adopting and implementing integrated policies and plans towards inclusion, resource efficiency, mitigation and adaptation to climate change, resilience to disasters, and develop and implement, in line with the Sendai Framework for Disaster Risk Reduction 2015-2030, holistic disaster risk management at all levels

13 CLIMATE ACTION

Goal 13. Take urgent action to combat climate change and its impacts

Climate change is one of the major triggers for preventive and corrective actions in hydraulic engineering, especially with extraordinary events that are becoming more and more frequent. This is why this objective is pursued, not only to mitigate the effects of the consequences of climate change, such as extreme events, but also to take climate change into account when carrying out projects and to try to establish sustainable practices to reduce it. The targets referred to in this work are 13.1, 13.2 and 13.3.

13.1 Strengthen resilience and adaptive capacity to climate-related hazards and natural disasters in all countries.

13.2 Integrate climate change measures into national policies, strategies and planning.

13.3 Improve education, awareness-raising and human and institutional capacity on climate change mitigation, adaptation, impact reduction and early warning.

15 LIFE ON LAND

Goal 15. Protect, restore and promote sustainable use of terrestrial ecosystems, sustainably manage forests, combat desertification, and halt and reverse land degradation and halt biodiversity loss.

Working on the benefits of sustainable drainage systems, besides considering the improvement in terms of water quantity and quality control, mention has been made of their favourable character in terms of ecosystems and biodiversity. Thus, the implementation of these systems pursues targets 15.3 and 15.a.

15.3 By 2030, combat desertification, restore degraded land and soil, including land affected by desertification, drought and floods, and strive to achieve a land degradation-neutral world

15.A Mobilize and significantly increase financial resources from all sources to conserve and sustainably use biodiversity and ecosystems

15 CONCLUSION

This study has demonstrated the satisfactory adaptation of sustainable drainage systems in industrial estates, obtaining benefits that can be even greater than those produced in urban areas. These different benefits in industrial estates have been presented, highlighting:

- The **reduction of runoff volumes and lamination** of hydrograms, ensuring the hydrological and ecological conservation of the receptor media and the mitigation of the risk of local flooding.

- **Efficient treatment of the pollutant loads** carried by rainwater, thus causing less impact on the water masses that receive these volumes.

- **Economic savings in the integral water cycle**. With them, the collectors that make up the urban drainage systems can be reduced in size, which leads to lower construction costs.

- Creation of many **ecosystem services**, which aim to favour and improve the lives of citizens and the living beings that make up the local biodiversity.

Furthermore, consideration was given to the selection of the most appropriate drainage systems for the situation, by evaluating different factors and determining the suitability of these systems in order to find the optimum solution. All of this is supported by municipal guides that back up the choice of determining the system that best adapts to the study site, as well as other international guides that serve as support for the development and implementation of certain SuDS.

In the specific case study, it has been shown that sustainable drainage systems are the optimal solution for the drainage system of the industrial estate of Quart de Poblet, Valencia, Spain. This assertion was based on a comparison of four different alternatives, varying traditional and sustainable development approaches and on simulation using hydraulic modelling tools. A multi-criteria analysis has been carried out, considering financial, hydraulic, energy, environmental and social aspects, demonstrating that, although these alternatives are more costly economically, their hydraulic response is more than satisfactory, coming close to ambitious hydraulic objectives and, of course, representing an improvement in terms of energy savings, Co2 equivalent emissions and general social and environmental benefits.

In view of all the work carried out, there is no doubt that sustainable drainage systems are a solution that should be studied as an alternative for new drainage system projects or, at the very least, considered for integration into existing drainage systems. Highlighting their effectiveness in industrial estates.

However, despite all the favourable results in this and other studies in the same field, there is still much work to be done to achieve widespread and standardized implementation of SuDS. To achieve this, two fundamental aspects will have to be worked on together: financing and regulation.

- **Financing.** The commitment to these sustainable drainage techniques does not usually require additional financial resources, although they do require more effective management of existing funds. The analysis of their profitability cannot be based solely on economic criteria and the hydrological results obtained but should also evaluate the benefits they provide in terms of landscape quality, adaptation and mitigation of climate change, and improvements in the well-being of the population, transforming them into monetary value.

- **Regulation.** It is important to have legal framework support for initiatives related to sustainable water management. Therefore, efforts should be made to promote these sustainable techniques in an efficient way through existing frameworks, so that they can be considered as a real drainage alternative.

Furthermore, work must be done to improve two parameters: **cross-sectoral collaboration**, whose improvement will facilitate change and decision-making, and the **knowledge base**, to raise awareness of the processes and benefits of SuDS through training and awareness campaigns, training professionals in design, implementation, maintenance and evaluation, and developing common general criteria for different water resources management options.

ANNEX Nº1: TECHNICAL FRAMEWORK. SUSTAINABLE DRAINAGE SYSTEMS

SUSTAINABLE DRAINAGE SYSTEMS (SUDS) IN INDUSTRIAL AREAS:
APPLICATION TO THE INDUSTRIAL AREA IN QUART DE POBLET (VALENCIA)

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Summary

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1 OBJECT

This annex aims to present the whole technical framework that involves sustainable drainage systems and which will serve to justify the proposed solution. It indicates the criteria, types, functions and advantages of sustainable drainage systems and adapts their implementation to industrial estates.

The conventional drainage system has been based, from the beginning, on immediately collecting all surface water runoff and carrying it out of the cities through drains and piping systems. This methodology was intended to provide a fast solution to the problem of water management.

With the passage of time, urbanisation and industrialisation developed at the same time as the land acquired more and more impervious surface, thus modifying the natural state of the land. The consequence of the imperviousness of this land is the alteration of the natural water cycle. With the development of urban areas, the water that, in its natural state, is retained and infiltrates into the ground, becomes part of the runoff that needs to be collected, conveyed and discharged into the receiving environment.

This process causes an increase in runoff volumes and peak flows, a decrease in the time it takes to reach the peak flow, a decrease in the base flow that ends up in natural channels and a decrease in the infiltration of water into the subsoil to recharge aquifers. In addition to these aspects, impermeabilization also means an increase in the pollution present in surface runoff, as a result of the sum of the pollution that descends from the atmosphere with precipitation and the pollution collected from permeable surfaces due to the "washing effect" of surface runoff. All this leads to a problem of stormwater runoff management due to the quantity and quality of the water discharged into the receiving environment.

To reduce the impact of urbanisation, and therefore impermeabilization, on drainage systems and discharges into the receiving environment, the implementation of Sustainable Drainage Systems, better known as SuDS, is used.

2 THE SUDS PHILOSOPHY

The SuDS philosophy is based on reproducing the natural water cycle in a natural state of the land, before impermeabilization. To this effect, the aim is to incorporate permeable systems that reduce surface runoff by achieving:
- Infiltration into the ground
- Lamination of flows
- Increase in the quality of water discharges.

Sustainable drainage must be approached as a complementary system to conventional drainage to maximize the smart performance of the drainage system. The approach addresses hydrological, environmental and social aspects. One aspect that characterizes SuDS is their relationship to the circular economy, a term that has been heard more and more recently at the international level. SuDS consider surface water as an asset to be reused, rather than as a waste to be disposed of as would be the case with conventional drainage. Another characteristic that distinguishes SuDS from conventional drainage systems is the fact that they act at source, during transport and at destination. In this way, by acting from the source, a decentralized system is achieved.

Sustainable drainage also acts to increase biodiversity and the creation of multifunctional spaces and landscape value for the residents.

The SuDS Manual (Woods Ballard, B. et al., 2015) sets out the benefits of SuDS in four broad groups:
- Water quantity,
- Water quality,
- Amenity
- Biodiversity.

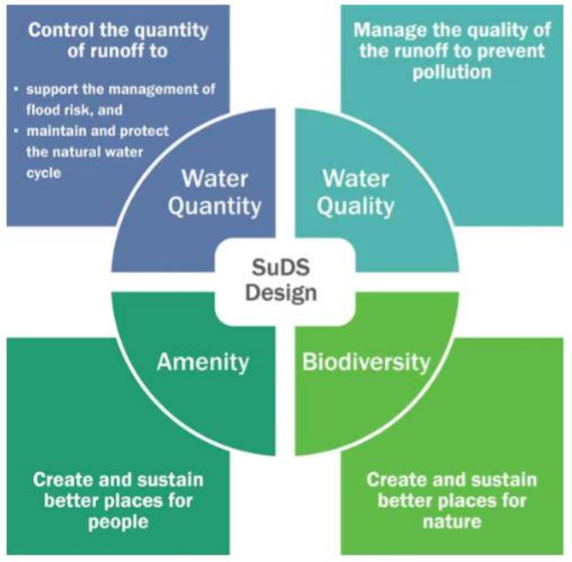


Figure 1.SuDS benefits. Source: Woods Ballard, B. et al., 2015

3 SUDS TYPOLOGY

Sustainable drainage can be identified according to two main groups:
- Non-structural: their implementation does not require any specific intervention on the drainage system.
- Structural: actions that require, to a greater or lesser degree, some constructive element.

In this study we are going to focus mainly on the implementation of structural sustainable drainage systems. However, later mention will be made of non-structural SUDS that will help in the management of surface runoff pollution.

3.1 Roles of structural SuDS components

The main roles of SuDS components are retention, detention, filtration, infiltration and treatment.

- **Retention**: Systems with retention components to capture stormwater and runoff to facilitate on-site utilisation for non-drinking water use.

- **Detention:** Systems with detention components to temporarily store runoff for progressive discharge while promoting particle settling and reducing peak flow.

- **Filtration:** Systems with filtration components with permeable surface to allow water to penetrate while reducing the amount of runoff to be carried through conveyance systems for treatment, with the possibility of underground storage and self-treatment.

- **Infiltration:** Systems with infiltration components to facilitate infiltration of water into the ground, with the possibility of temporary storage of water to reduce runoff volumes at peak flows.

- **Treatment:** Systems with treatment components to remove degradation, sediment and pollutants that may be present in runoff.

3.2 Type of structural SuDS

The following table presents the division of each type of structural SuDS according to their primary and secondary function and their suitability for deployment in industrial areas.

Table 1. Classification of structural SuDS according to function and suitability for industrial áreas. Source: Perales, S. et al., 2019

SuDS typology	Primary function	Secondary function	Use at industrial premises
Trees	Infiltration	Detention	Yes
Infiltration trenches	Infiltration	Filtration	Yes
Reticular tanks	Infiltration	Detention	Yes
Pervious pavements	Filtration	Infiltration	Yes
Filter drains	Filtration	Detention	Yes
Rain gardens	Treatment	Infiltration	Yes
Bioretention systems	Treatment	Infiltration	Yes
Swales	Treatment	Infiltration	Yes
Wetlands	Treatment	Detention	Yes
Ponds	Treatment	Detention	Yes
Detention basins	Detention	Infiltration	Yes
Infiltration basins	Detention	Infiltration	Yes
Green roofs	Retention	Detention	Maybe
Attenuation storage tanks	Retention	Detention	Yes

The choice of the type of sustainable drainage in the industrial area will be made later, explaining which of them are the most suitable for this type of surface due to their configuration. Before doing so, it is considered useful to briefly describe the types of structural SuDS that could be implemented, to better understand how they work.

- **Trees:** formed by an excavation in the ground where a tree is planted with a structural soil backfill. The backfill can be made of granular material or polypropylene geocellular material. It temporarily stores runoff allowing tree roots to grow.

- **Infiltration trenches:** they intercept runoff and temporarily store it on the subsurface for later infiltration into the ground. It is an excavation in the ground filled with draining material with a high percentage of voids, which can be granular or synthetic.

- **Reticular tanks:** underground structures that collect runoff to store it temporarily and later infiltrate it into the subsoil or release it slowly. It is created with reticular polypropylene structures with a high void ratio and high bearing capacity.

- **Pervious pavement:** pavements that filter runoff to lower layers. Water is temporarily stored in the sub-base composed of gravels or reticular boxes before infiltration into the ground or controlled discharge through drains.

- **Filter drains:** shallow trenches filled with filter material with a drain at the base. Runoff is filtered and temporarily stored and then discharged downstream through drains.

- **Rain gardens:** vegetated area lowered to store surface runoff that is filtered before infiltration. It has a filter medium to filter and absorb pollutants. If the soil is not sufficiently permeable, a drain can be installed to discharge runoff downstream.

- **Bioretention areas:** like rain gardens, but with a thicker layer of filter media and used to manage runoff with high levels of pollutants.

- **Swales:** wide, shallow, linear structures covered with vegetation to capture, filter and infiltrate in some cases. Runoff drains evenly and directly down the sides, minimizing erosion and distributing pollutants.

- **Wetlands:** they improve quality by settling pollutants. When the sheet of water exceeds storage, the runoff is detained and slowly discharged downstream through an overflow structure.

- **Ponds:** Like constructed wetlands, but with a greater depth, less vegetation cover and a higher pollutant mitigation rate.

- **Detention basin:** vegetated depression that is usually dry but offer temporary surface storage when it rains to release runoff later providing peak flow lamination. They laminate flows with a controlled bottom outflow.

- **Infiltration basin:** Like detention basins, but instead of releasing runoff through a controlled bottom drain, they allow runoff to be evacuated through soil infiltration.

- **Green roofs:** a vegetated package installed on the roof surface, consisting of vegetation on a substrate, geotextile filter sheet, drainage system and waterproof membrane underneath.

- **Attenuation storage tanks:** capture runoff for later use. Runoff is stored, treated if required and can be used for non-potable water uses. They can be prefabricated polyethylene or in-situ concrete.

The different types of structural sustainable drainage systems perform differently in terms of peak flow control, volume control and quality control. Their classification is shown in the following table identifying those that work most efficiently:

Table 2. Classification of structural SuDS according to performance. Source: Perales, S. et al. (2019)

SuDS typology	Peak flow control	Volume control	Quality control
Trees	Medium	Good	Good
Infiltration trenches	Very good	Very good	Good
Reticular tanks	Very good	Good	Medium
Pervious pavements	Very good	Very good	Very good
Filter drains	Good	Medium	Good
Rain gardens	Good	Good	Very good
Bioretention systems	Good	Good	Very good
Swales	Good	Medium	Very good
Wetlands	Good	Medium	Good
Ponds	Good	Medium	Good
Detention basins	Very good	Good	Good
Infiltration basins	Very good	Good	Good
Green roofs	Medium	Good	Medium
Attenuation storage tanks	Medium	Good	Limited

4 SUDS IMPLEMENTATION AT INDUSTRIAL PREMISES

The advantages of using sustainable drainage in urbanised areas generally have been explained. The specific case of application of the study focuses on industrial areas, and it will therefore be shown why the use of this type of system can improve the drainage system in industrial estates where industrialisation has taken place. The aim is to demonstrate that the use of sustainable drainage in industrial areas has a positive impact.

It is important to bear in mind that the analysis of SuDS in an industrial area goes a step further than the implementation of SuDS in a residential urbanised area. This statement is based on the principles described above on the philosophy of SuDS. Primarily, the implementation of SuDS is done to manage the volume of runoff generated, which is mitigated by the contingency functions of SuDS. However, at the same time, SuDS implementation manages the quality of this runoff. This last point is of vital importance for industrial areas as they are in an environment where the risk of runoff water pollution is greater, generating a lower quality of runoff water.

4.1 Pollution impact

To analyse pollution in industrial areas, it is necessary to differentiate between wastewater pollution and stormwater pollution. Wastewater is used, domestic, urban and industrial wastewater and liquid industrial waste, while stormwater is runoff water from rainfall events.

In the field of industry, it is currently underway to develop green industry that offers progressive integrated approaches to solving effluent pollution problems. Most of the industry is in industrial estates and relatively few industries discharge their wastewater directly into the receiving environment. However, this does not mean that there is no pollution discharged into the receiving environment. Many of these industrial sites are located on large impervious surfaces, where surface water drainage and stormwater runoff is often polluted from a number of different causes. In this regard, studies have quantified the environmental problems and identified the best practical solutions to mitigate this problem, putting in place recommendations that provide effective means to prevent pollution and achieve good working environments in which industry can work.

In this regard, some studies have quantified environmental problems and identified the best practical solutions to mitigate this problem, putting in place recommendations that provide effective means to prevent pollution and achieve good working environments in which industry can thrive.

It is important to highlight the positive impact of sustainable practices in industrial areas because it is not only the environment that benefits from these practices. These techniques have a positive impact on the economy. Intelligent site design is an opportunity not only to benefit the environment, but also to limit the impact on clean, unpolluted water. When an industry pollutes a river, it is not only an ecological problem, but there are also economic impacts on communities, becoming widespread economic impacts if a country or region is known to be polluted (D'Arcy, B. et al., 2017).

4.2 Pollution risks at industrial premises

The design of a drainage system in industrial areas requires the consideration of two categories for the identification of pollution risk. Firstly, the contingency and risk management plan and secondly, the design for the capture and dissipation of pollutants in runoff.

Several studies have independently demonstrated that SuDS not only capture and dissipate pollutants, but also provide "in situ" degradation. An example of this is the study of the behaviour of pollutants within SuDS structures, beyond the retention of polluted solids by filtration and sedimentation due to traffic, where the behaviour of heavy metals, oils and hydrocarbons was observed in southern Scotland (Napier, F. et al, 2009).

In the following table it can be seen how the different SuDS can be important elements for strategic management in industrial development can be. One aspect that is added to the table beyond water quantity and quality management is the high visibility of clean-up identification. This aspect is important to identify especially hydrocarbons and fats that may be present in runoff from industrial areas and that would be difficult to identify with a conventional system.

Table 3. Utility of SuDS at industrial premises. Source: : D'Arcy, B. et al., 2017

SuDS typology	Pollutant degradation	Storage capacity	High visibility of clean-up identification
Trees	x	x	
Infiltration trenches	x		
Reticular tanks		x	
Pervious pavements	x	x	
Filter drains	x		x
Rain gardens	x		x
Bioretention systems	x		x
Swales	x		
Wetlands	x	x	x
Ponds	x	x	x
Detention basins	x	x	x
Infiltration basins	x	x	
Green roofs	x		
Attenuation storage tanks		x	

The location of SUDS should be determined at the source, at the pipeline and at the end of the system. Control SUDS are needed at the source to capture pollutants as close to the emission source as possible.

4.3 Pollutant treatment management chain

The treatment capacity of SuDS in an industrial area must consider two factors:
- The collective plan that aims to produce quality water for river drainage, developed as a package.
- The types of treatment processes that are present in each SuDS component of the infrastructure.

The first treatment factor, the collective plan, is the sum of all treatment processes in each SuDS component. The treatment capacity of SuDS is influenced by how a component will capture pollutants that exist in the runoff, as well as the destination of the captured pollutants. Also, it is important to determine what proportion of the pollutant in the runoff will be treated and what proportion will pass through.

A treatment chain allows the sequential capture of pollutants in successive SuDS components, the different characteristics of each SuDS in pollutant capture or degradation of pollutant properties are key to the incorporation of each type into the treatment chain. The cumulative removal of pollutants contained in runoff water through SuDS will enhance the importance of biodiversity and amenity in any of them at the end of the system.

The stormwater management chain seeks to include all aspects that are necessary for management. This means that it not only focuses on the risk of pollution, but also considers the risk of flooding. In its objectives, it seeks to attenuate the flow of runoff, while adding treatments for stormwater runoff. It incorporates aspirations characteristic of the treatment chain in a wider context. The management of surface water runoff quality to protect the receiving environment, as well as the effectiveness of treatment to prevent pollution, is strongly linked to the hydraulic control of runoff. In particular, the following parameters:
- **Velocity control**: removal processes occurring at low outflow velocities during rainfall events.
- **Retention time**: the removal of pollutants that occur during the period that the runoff is in contact with the SuDS treatment media or is held within a water storage volume.

Different methods can be found in the literature for the determination of the hazard posed by different land uses and the extent to which the underlying soil layers and proposed treatment components reduce the risk of contamination. The different methods can generally be summarized as follows:
- Simple qualitative methods
- Established risk screening methods
- Detailed risk assessment methods
- Process-based treatment modelling methods

In this paper, we will further elaborate on the proposed pollution level index method as a simple qualitative method (Woods Ballard, B. et al., 2015). The steps for this method are:
- Assign appropriate pollution hazard index for the proposed land use.
- Select SuDS with a total pollution mitigation index that equals or exceeds the pollution hazard index.
- Where the discharge is to protected surface water or groundwater, consider the need for a more precautionary approach.

The pollution hazard index varies according to the land use to be considered from 0 to 1, with 1 being the maximum level of pollution and 0 being no pollution at all. In the case of land in industrial areas, a high contamination risk level is considered with the following contamination index:

Table 4. Pollution hazard index at industrial premises. Source: Woods Ballard, B. et al., 2015

Total suspended solids	Metals	Hydrocarbons
0,8	0,8	0,8

If the use of a single SuDS component is not sufficient, two or more components may be placed in series, forming this chain of treatments, considering that the mitigation index shall be:

$$Total\ mitigation\ index = Index\ mitigation\ 1 + 0{,}5 * Index\ mitigation\ n$$

The factor 0.5 is used to reduce the effectiveness of SuDS components in secondary and tertiary series, because the incoming runoff contains a reduced concentration of pollutants.

5 POLLUTANT REMOVAL MECHANISMS BY SUDS

In addition to outlining the main mechanisms by which SuDS work: retention, detention, filtration, infiltration and treatment, this section aims to present several water quality treatment processes that can be used in the design of a sustainable drainage system for the removal and mitigation of pollutants.

5.1 Sedimentation

Sedimentation is one of the main mechanisms for the elimination of SuDS. The main reason for this is because most of the pollution in runoff is attached to sediment, therefore the removal of sediment from runoff leads to a significant reduction in pollutant loading.

This process is achieved when outflow velocities are low and suspended sediments can settle out. This process requires periodic removal to allow for non-clogging of the SuDS. In addition, it should be noted that the design of SuDS should minimise the risk of resuspension during extreme rainfall events.

5.2 Filtration and bioretention

Some of the pollutants that are transported in sediments can be filtered out. This occurs when they are trapped in the soil or aggregate matrix, in plants or in geotextile layers. When designing SuDS a balance must be achieved between the efficiency of pollutant mitigation and the potential risk of clogging of the filtration component.

5.3 Separation

Separation attempts to remove pollutants that float on the surface of the water as an immiscible layer or can be captured by vegetation, allowing their further degradation, through volatilization or photolysis. This type of mechanism is used for the removal of many hydrocarbons and some other pollutants, such as pollen.

5.4 Adsorption

Adsorption is a mechanism that seeks to remove contaminants by adhering them to the surface of aggregates, soil, sand or artificial material. The process tends to be a combination of complex surface reactions, which depend on the acidity of the runoff to increase or decrease.

The design must take into account that the materials to which the contaminants adhere will become saturated, so this mechanism must have periodic maintenance and regeneration.

5.5 Biodegradation

Biodegradation is a treatment different from the other physical and chemical processes. It is a biological treatment in which micro-organisms in the soil and aggregate matrix use oxygen and nutrients from the input materials to degrade organic pollutants in the runoff, such as oils and fats. The level of efficiency of this mechanism depends on environmental conditions such as temperature, oxygen and nutrient supply, as well as the physical suitability of the materials for colonisation by the micro-organisms.

5.6 Volatilisation

La volatilización implica la transferencia de un compuesto de la fase sólida o líquida a la atmósfera. La conversión a un gas o vapor se ve influida por la temperatura, la reducción de la presión, la reacción química o una combinación de estos procesos. En los SUDS, la volatilización se refiere principalmente a los compuestos orgánicos asociados a los productos del petróleo y a los pesticidas.

5.7 Precipitation

Precipitation is the most common mechanism for removing soluble metals. It is a chemical treatment between contaminants and compounds in the soil or aggregate matrix that transforms the dissolved constituents into insoluble precipitate particles. Metals precipitate in the form of hydroxides, sulphides and carbonates, depending on the precipitants present and the pH level.

5.8 Absorption by vegetation

Plant absorption is an important removal mechanism for nutrients such as phosphorus and nitrogen, specifically by creating biofilm around the plant structure. Metals can also be removed by this mechanism, however, periodic maintenance must be necessary to remove the plants, as plant death could result in metals being returned to the water. In addition, plants also create suitable conditions for the deposition of metals, such as sulphides, in the root zone and provide a microbiological environment that favours the biodegradation of organic pollutants.

5.9 Photolysis

The photolysis mechanism is a process based on the breakdown of surface organic contaminants by exposure to ultraviolet light.

5.10 Hydrolysis

Hydrolysis also plays an important role, being the chemical decomposition of an organic compound due to reaction with water.

5.11 Oxidation

Oxidation is the chemical process that allows the combination of a compound with oxygen through the loss of electrons.

5.12 Reduction

Reduction is the opposite of oxidation. It is a chemical reaction in which the compound gains electrons through the loss of oxygen.

5.13 Substitution

Substitution is a chemical reaction in which the functional group in one compound is replaced by another, thus changing its structure.

6 POLLUTANTS IN RUNOFF FROM INDUSTRIAL AREAS

In most instances, when reference is made to municipal stormwater, it is directly associated with surface runoff in urban centers. This runoff is collected in the drainage system and discharged to the receiving medium, be it a river or a sewage treatment plant. Along the way, the runoff picks up all kinds of pollutants that can pollute the receiving environment or overload wastewater treatment plants. Already a potentially significant problem, stormwater pollution increases enormously in industrial areas. In the case of industrial areas, pollution becomes a point source due to the contact of manufacturing materials with rainfall falling on the site.

Industrial facilities are generally located in urbanised areas with large impervious surfaces and operate large machinery, creating a scenario that would resemble that of a residential area but much more concentrated, with higher pollutant levels and even pollutants that would not normally be found in a residential area.

The United States is probably one of the pioneers in establishing legislation to regulate rainwater in industrial areas and is very aware of the problem of runoff water quality.

The US Environmental Protection Agency (EPA) requires a permit, called NPDES or MSGP, depending on the case, in order to discharge stormwater from industries. Thus, the Clean Water Act prohibits the discharge of pollutants, including surface runoff, into US water unless a permit is obtained. These permits contain limits on what can be discharged, monitoring and reporting requirements, and other provisions to ensure that the discharge does not impair water quality or compromise human health. In general, the permits indicate general requirements adapted to the operations of each industry and its characteristics.

Thus, federal regulations identify 11 categories of stormwater discharges associated with industrial activity that must be covered by the NPDES permit. Pollutants in runoff from industrial areas vary depending on the industry. This is because each industry carries out processes that involve different materials and may add more or less pollutants of each type. Therefore, depending on the industry, different levels will be required according to their characteristic pollutants.

6.1 Common pollutants and prevention methods with non.structural SuDS

To better illustrate the mitigation indexes, it is important to know the most common pollutants from runoff from industrial areas, which are listed in this section (West, M et al., 2015):

6.1.1 Total suspended solids

Total suspended solids are particles and debris that have been washed off surfaces. The problems associated with suspended solids affecting surface water include affecting aquatic life, causing turbidity that impairs the eyesight of fish and increasing the cost of wastewater treatment. In addition, they act as a vehicle for transporting other pollutants such as nutrients and organic matter to surface waters.

Techniques to reduce suspended solids in stormwater runoff with sustainable non-structural drainage are:

- Good maintenance with measures such as frequent sweeping of outdoor areas.
- Addition of fibre logs or rock filters upstream of existing green areas to reduce water velocity.
- Adding fibre or synthetic geotextiles in eroded non-vegetated areas.

6.1.2 Metals

As metals corrode, they dissolve or settle in the air and small amounts are washed away by wind or water and become concentrated in stormwater runoff. Most of these metals adhere to sediment particles and are carried into the receiving environment. When these sediments settle, the attached metals accumulate over time to concentrations that are harmful to life. Even some metals in sediments can accumulate in plants and aquatic life. Mainly the metals found are aluminium and zinc, but many others such as lead, copper, etc., can also be found.

Techniques to reduce metal levels are:

- Control at source by limiting the exposure of metals to stormwater.
- Modifying processes, storage or handling.
- Minimising or eliminating the use of metals contained in production processes.
- Replace or paint galvanised surfaces.
- Use non-rubber equipment tyres.
- Add recycling to recover and recycle specific metals from production processes.

6.1.3 Hydrocarbons and petroleum products

Hydrocarbons are of concern because many of them are known to be toxic to aquatic organisms at relatively low concentrations. Several types of hydrocarbons are commonly found in industrial areas, mainly naphthalene, phenol, oil and grease. These pollutants are highly mobile and can exist for long periods in a toxic state. In addition, they can concentrate in sediments to return to suspension later. Oil contributes to significant water quality problems if not properly managed.

Petroleum-derived hydrocarbons commonly found in stormwater runoff often float on the water surface creating a sheen film that facilitates their recognition.

Non-structural techniques to reduce hydrocarbon levels include:

- Tank surface washing.
- Indoor maintenance and storage to remove contaminated materials that meet stormwater runoff.

6.1.4 BOD and COD

It refers to the amount of oxygen needed to chemically (COD) or biologically (BOD) decompose micro-organisms in water.

Aquatic life depends on the oxygen dissolved in the water. High concentrations of BOD and COD imply high oxygen requirements to break down matter carried by runoff. These requirements would be removing the available oxygen in the water, which is necessary for aquatic life, causing the death of living things consequently.

Techniques to reduce the BOD and COD levels of runoff with non-structural sustainable drainage are:

- Erosion control.
- Waste management and prevention.
- Protected and contained loading and unloading practices.

6.1.5 Nutrients

Excessive nutrients in the water can lead to algal blooms or other conditions toxic to aquatic life. The main problem nutrients are phosphorus and nitrogen.

High phosphorus contents lead to the lagooning effect in which there is excessive algal growth, oxygen depletion of the water and accelerated sedimentation of the lakes when the algae die.

Sustainable non-structural drainage techniques to reduce phosphorus levels are:

- Clean materials from permeable surfaces.
- Covering raw material and waste piles.
- Store materials indoors where possible.
- Capture and treat waste streams separately.
- Reduce water velocity to allow attenuation of nutrients by green areas before run-off occurs.

High nitrogen contents affect both groundwater and surface water as they present a health hazard.

Techniques to reduce nitrogen levels are:

- Control at source by implementing fertiliser application limits.
- Minimising or eliminating exposure prior to discharge.
- Maintenance and clean-up such as sweeping of solid spills.

A particular case of nitrogen is the formation of ammonia ($NH3$) which causes the pH to drop to background levels and the formation of nitrate and nitrite by nitrification. This leads to the consumption of high oxygen contents affecting aquatic life.

Techniques to reduce ammonium levels are:

- Control at source with the use of non-ammonia based cleaners.
- Minimising or eliminating exposure prior to discharge.
- Maintenance and clean-up such as sweeping of solids spills.

6.1.6 Chlorides and salts

Most of the chlorides come from ordinary salt (sodium chloride), generally the salt found in water is due to the salt used to melt ice in the winter season. Most of the salt used on roads ends up in stormwater runoff. When salt runoff meets the aquatic environment, it increases the potential toxicity of many freshwater organisms. In Spain, the use of salt for snowmelt is not as common as it may be in other colder countries. The risk of salts in runoff is much lower than in northern countries. However, it should be considered for specific cases.

Non-structural techniques to reduce chloride and salt levels are:

- Adequate storage.
- Sweeping and proper removal of any visible salt deposits after the application surface has dried.
- Proper sweeping and removal of any visible salt deposits at the end of the de-icing season.
- Use of chemical or special filters to remove dissolved salts from storm water before it leaves the facility

6.1.7 Temperature changes

Temperature changes can be a shock to living beings in the aquatic environment. The most common reasons for temperature rise are installations with large impermeable surfaces and large retention ponds used for stormwater treatment. This parameter must be considered when runoff is to be discharged into temperature sensitive waters such as a stream or fish lake or in cases where the discharge is excessively heated.

Techniques to reduce temperature rise include:

- Disconnect drainage pipes that discharge onto impervious surfaces and divert the water to green areas.
- Direct roof drains to green areas.
- If retention ponds are used, use filtered bottom water for outfall.

6.1.8 Turbidity

Turbidity is the most common cause of total suspended solids and dissolved materials. The most direct way to reduce turbidity is by reducing total suspended solids using the techniques mentioned above.

6.1.9 pH

A deviation from neutral pH in both acidity and basicity can be toxic to aquatic life and can cause other detrimental problems such as dissolution of concrete and metal pipes. In addition, extreme pH values, acidic or basic, can cause skin burns or eye irritation. Once runoff has become contaminated, treatment usually requires the addition of chemicals to neutralise the mixture and bring the pH to an acceptable range.

Non-structural techniques to reduce pH variation are:
- Reduce sources of pH-related chemicals.
- Apply pH-related chemicals at the lowest possible rates.

6.2 Mitigation of pollutants with structural SuDS

In the previous section, the most common pollutants in runoff water from industrial areas and how to prevent them with non-structural SuDS techniques were presented. In this section, we will discuss the mitigation of runoff pollutants with structural SuDS, presenting the mitigation indexes of each structural element to know which will be more effective in reducing pollution.

Before this, it should be noted that the risk of surface water runoff to the receiving environment is a function of:
- The pollution hazard at a given location, i.e., the source of pollution.
- The effectiveness of the SuDS components in mitigating pollutant levels to environmentally acceptable levels, and the effectiveness of the underlying soil layers in protecting the receiving groundwater.
- The sensitivity of the receiving medium, i.e., the environmental receptor.

Table 5. Mitigation indicators for SuDS in surface water discharges. Source: Perales, S. et al, 2019

SuDS typology	Total Suspended Solids	Metals	Hydrocarbons
Trees	0,6	0,5	0,6
Infiltration trenches	0,4	0,4	0,4
Reticular tanks	-	-	-
Pervious pavements	0,7	0,6	0,7
Filter drains	0,4	0,4	0,4
Rain gardens	0,6	0,5	0,6
Bioretention systems	0,8	0,8	0,8
Swales	0,6	0,6	0,6
Wetlands	0,8	0,8	0,8
Ponds	0,7	0,7	0,5
Detention basins	0,5	0,5	0,6
Infiltration basins	0,6	0,5	0,6
Green roofs	0,4	0,4	0,4
Attenuation storage tanks	-	-	-

It is important to note that these mitigation indexes are not invariant. They will always be conditioned by the design of the SuDS and the materials used.

6.2.1 Complementary elements for pollutants' mitigation

In addition to SuDS in industrial areas, in some cases, complementary elements such as hydrodynamic separators and compact filters are used. Although these elements are not considered as structural sustainable drainage systems by themselves, they are an enabling factor to increase the performance and efficiency of SuDS.

Hydrodynamic separators are devices capable of removing settleable solids, floating debris, flat and hydrocarbons from runoff. Through hydrodynamic vortex separation, settleable solids settle to the bottom and floating debris, fats and hydrocarbons rise to the surface by suspension.

The benefits of hydrodynamic separators are manifold, including the following:
- Improves runoff water quality.
- Reduces maintenance and service life of structural SuDS by preventing clogging.

Compact filters are devices that remove pollutants present in runoff through filtration technologies. Runoff is filtered by an upward flow through inclined screens and filter modules. In this way, coarse solids and sediments precipitate at the base of the chamber, while grease and hydrocarbons rise to the surface. The benefits of compact filters include:
- High performance in improving runoff quality.
- Particularly suitable for industrial áreas

7 SUITABILITY AND LIMITATION OF SUDS IN INDUSTRIAL AREAS

The impact of sustainable drainage in industrial areas is not a subject that is widely represented in the Spanish literature. As indicated above, some articles and documents can be found that represent case studies, mainly from the United Kingdom and the United States, where the SuDS philosophy is more developed and implemented.

Among the texts that can be found, there are few that refer specifically to industrial areas. One example is the study carried out to determine whether stormwater runoff from an industrial area in the city of Fresno, California, had caused or had the potential to cause groundwater quality degradation (Schroeder, R. A., 1995).

Another more recent example is provided by the Houston Industrial Estate on the case of the discharge of runoff from an industrial area into the Caw Burn, which is known to have water quality problems (Krivtsov, V. et al., 2019).

In this section, the most suitable SuDS for industrial areas are going to be presented considering their structural and performance limitations.

Before going into detail, it is worth noting the importance of knowledge of the geology and geotechnics of the study area for the correct implementation of SuDS, mainly due to their infiltration characteristics. Knowing the properties of the soil is necessary to know the efficiency of SuDS and greatly conditions their design. Furthermore, not only for the infiltration properties themselves, knowing the permeability of the material allows us to know if there is the possibility of finding an impermeable clay layer, whose permeability could influence the direction of flow.

The risk of such situations lies in the fact that water flowing under the adjoining buildings could cause structural problems in the substructure and foundations. Therefore, beyond the suitability of SuDS in an industrial area as proposed in this work, in-depth studies on topography, geology and geotechnics should be carried out.

7.1 Roadways and parking facilities

One of the main factors determining the choice of SuDS in industrial areas compared to residential urbanised areas is the existence of heavy vehicles. Heavy vehicles condition the selection of drainage systems, excluding those types whose structural design or material properties cannot withstand the stresses generated, becoming damaged and requiring continuous repairs. This mainly affects SuDS arranged on the roadway and in parking areas.

Parking areas pose a risk of potential fluid spillage from poorly maintained vehicles, an example would be oil spillage from lorries and heavy vehicles, as well as from private cars. The choice of SuDS best suited to driveways and parking areas are **pervious pavements**. When the permeability of large areas, such as roadways, is required, porous asphalt (PA) is mainly used. This, by infiltrating runoff, allows for an attenuation of water going into the sub-layers of the ground. This method captures pollutants and reduces the risk of transferring these pollutants into the drainage system or into the groundwater. In the case of car parks, pervious block paving (PBP), which provides a permeable surface, is widely used. The performance of this type of pavement is like that of pourous asphalt, allowing infiltration and attenuation.

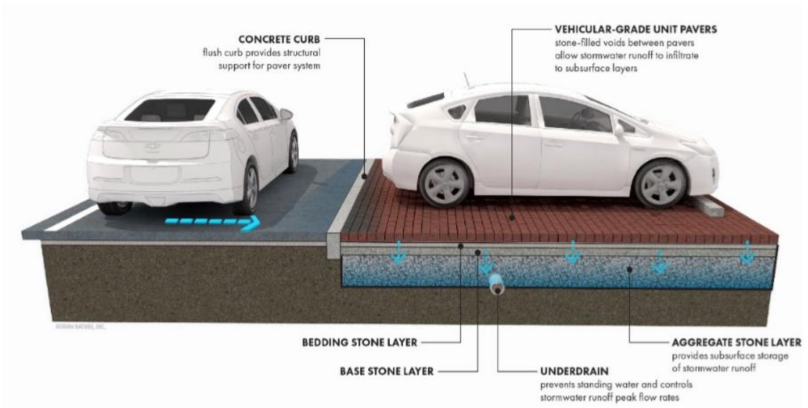


Figure 2. Scheme of pervious pavement. Source: University of Cincinnati, 2016

A critical point that must be considered always is the traffic that the pavement will have to support. Industrial areas are characterized by heavy traffic which, although they do not circulate at high speeds, their weight greatly conditions the pavement design. In areas where heavy vehicles pass frequently, such as delivery yards, pervious pavements cannot be installed as there is a risk of being compacted by heavy vehicles, requiring repair and additional costs. The use of these pavements will be destined for car parking.

In large impermeable areas where heavy traffic limits the use of pervious pavements, other solutions are proposed, such as redirecting rainwater to a nearby green area so that the runoff does not go directly into the drainage system and cause the disadvantages mentioned above. This can be achieved by installing **infiltration trenches, filter drains or vegetated swales** on the sides of the road to direct the runoff water to a green area treated with SuDS, such as a bioretention area, detention and infiltration basins or artificial wetlands and ponds if the available surface area allows it. This will filter and/or infiltrate the water and capture pollutant particles and fluids.

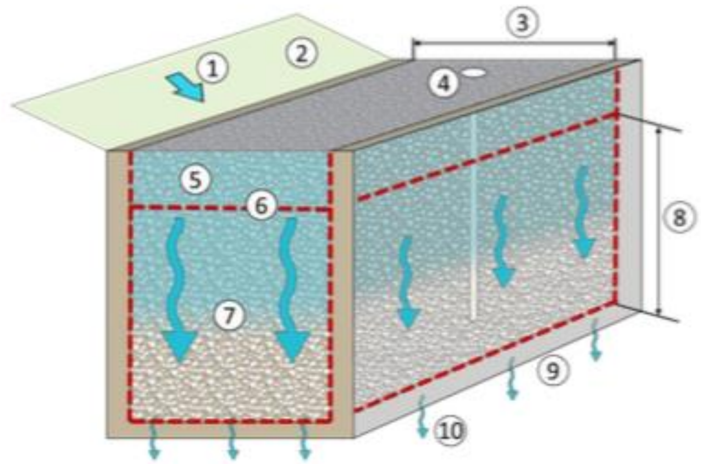


Figure 3.Filter trench scheme. Source: Perales, S. et al., 2019

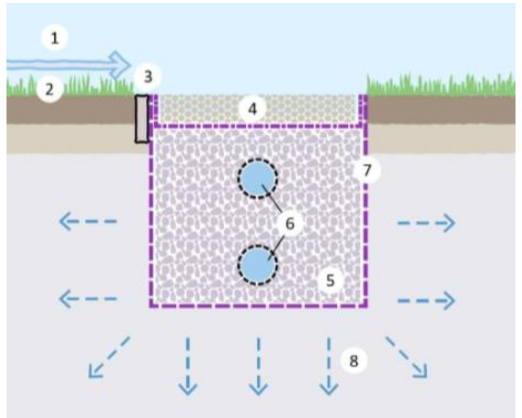


Figure 4. Filter drain scheme. Source: Perales, S. et al., 2019

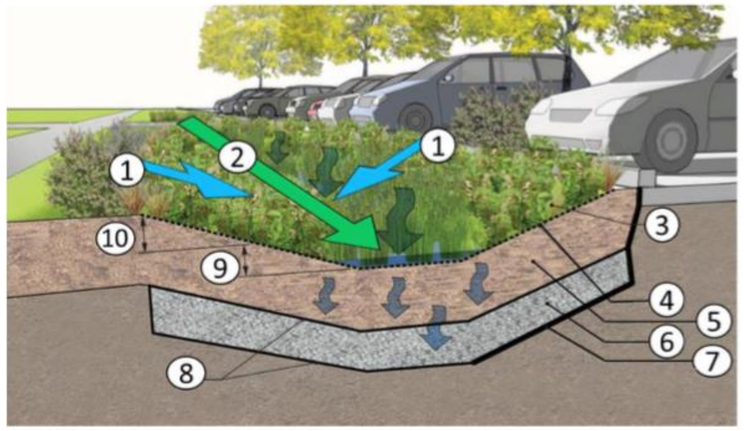


Figure 5. Swale scheme. Source: Perales, S. et al., 2019

7.2 Green areas

The limitations of green areas are mainly indicated by the available surface area. Ideally, any industrial area should have green areas determined in the General Urban Development Plan (PGOU), so that it can be used as SuDS.

Where green area space allows, **rain gardens** and preferably **bioretention areas** should be provided as they have a higher pollution mitigation rate. The emptying of both should always be ensured within 48 hours. **Trees** could also be implemented, but their limitations in terms of volume management would make the first two more suitable.

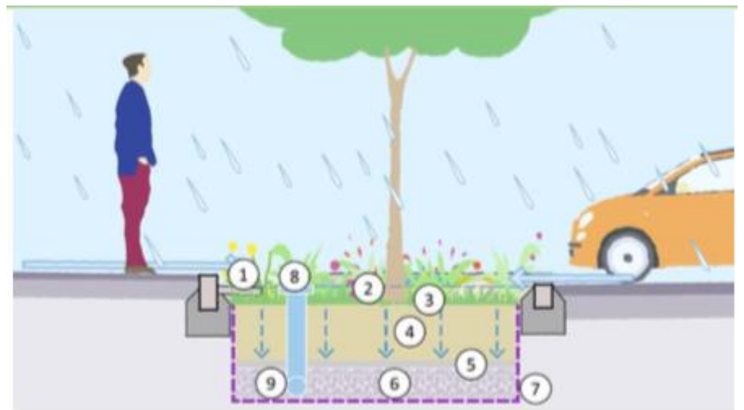


Figure 6. Rain garden scheme. Source: Perales, S. et al., 2019

Figure 7. Bioretention área scheme. Source: Alameda County Works Agency, 2019

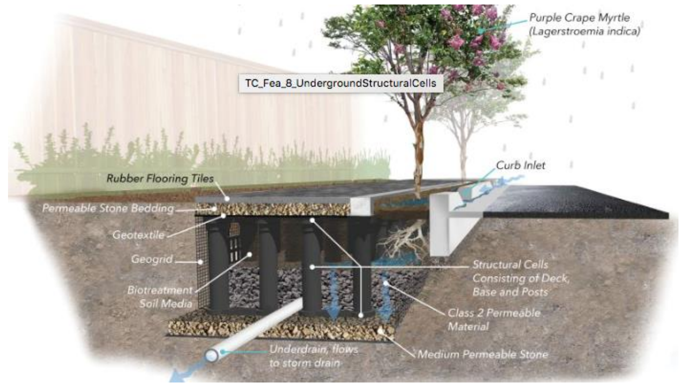


Figure 8. Tree scheme. Source: Alameda County Works Agency, 2019

Artificial **wetlands** and **ponds** are also a good option in the case of large green areas. It is recommended that the runoff has gone through a previous process because otherwise there would be an excess of sediment input that would clog them. Ponds are preferably used for water quantity control. In the case of seeking to give more importance to the mitigation of pollutants, artificial wetlands will be used. In the case of a green area near a road, other types of systems will be used.

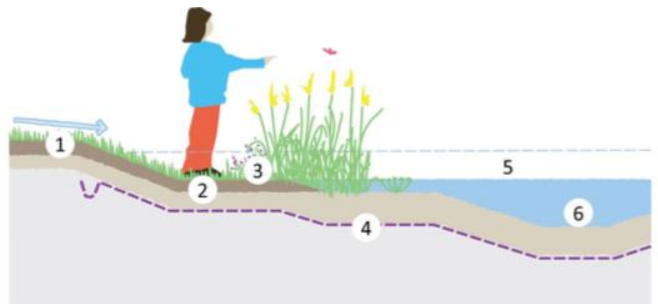


Figure 9. Wetland scheme. Source: Perales, S. et al., 2019

Detention basins and **infiltration basins** are also a good option when large areas are available. They have lower mitigation rates but offer the advantage that they only hold water mass in rainfall events, as they must be emptied in less than 48 hours and the rest of the time, they will remain drained. The fact that they are not continuously filled with water presents an advantage compared to constructed wetlands and ponds, which should be considered when implementing them in industrial areas, since if the water body is close to a road it can cause security and safety problems.

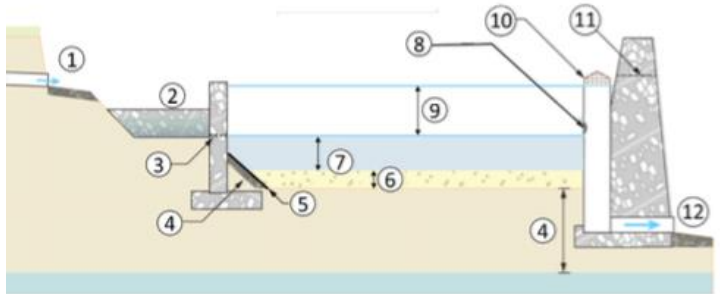


Figure 10. Infiltration basin scheme. Perales, S. et al., 2019

7.2.1 Roof

Conventional drainage systems on the roofs of facilities are usually downpipes that are directly connected to the network. However, the likelihood of them contributing significant contamination to the network is low, because, in general, the possible contamination would be from leaves from nearby trees or atmospheric pollution from rainfall.

The presence of these pollutants in the drainage system does not have a significant impact on the environment. However, it should be noted that these pollutants can affect the efficiency of the system by causing blockages or clogging.

One SuDS option for roofs is **green roofs**. The implementation of green roofs must be well considered as it has to be economically cost-effective and structurally feasible. In new industrial buildings it could be an option to be studied. However, in existing buildings where SuDS are to be installed, the costs of fitting out the structure to install a green roof that is structurally resistant are expensive and these fitting out works could have a significant impact on the day-to-day running of the industry.

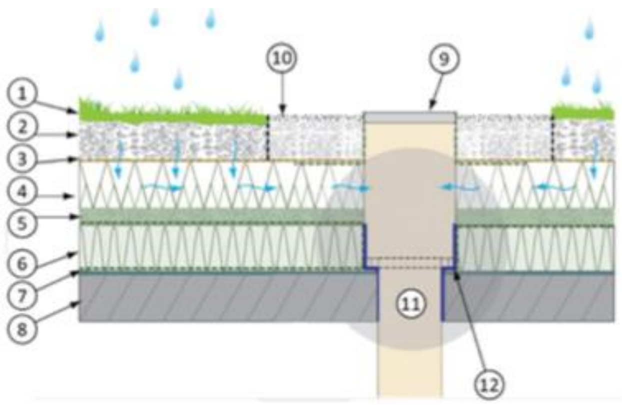


Figure 11. Green roof scheme. Source: Perales, S. et al., 2019

In these cases, there are other options for the implementation of SuDS. One of them could be the disconnection of the downpipes from the drainage system and their passage through reticular tanks, attenuation storage tanks or green areas such as rain gardens or others.

Reticular tanks are underground structures that allow drainage over a longer period, if they are made of semi-impermeable material, or control when and how much water is discharged, if they are covered with an impermeable material. In this case it should be noted that the pollutant mitigation capacity of the tanks themselves would be extremely low, so they would mostly serve to help with water quantity management.

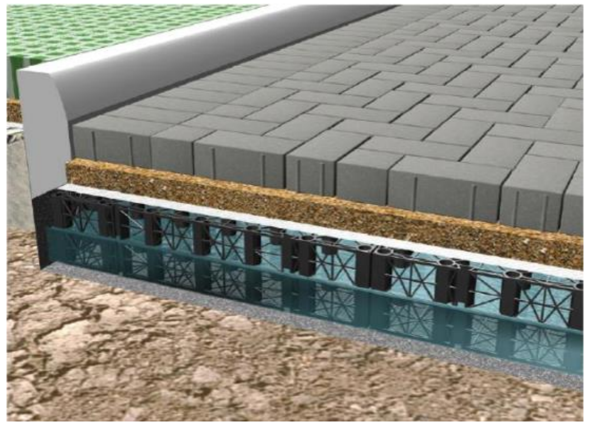


Figure 12.

Figura 1. Reticular tanks scheme. Source: Woods Ballard, B. et al., 2015

Attenuation storage tanks are above ground structures and would store runoff from the roof. This stored water could then be discharged into the ground or injected into the drainage system in a controlled manner. Cisterns are more expensive but cost less to maintain and install because they do not require excavation.

Figure 13. Attenuation storage tanks scheme. Source: Perales, S. et al., 2019

If there is a green area near the roof, **rain gardens** could be used, which would not only solve the problem but also improve the aesthetic appearance of the area. Rain gardens are mainly chosen among green areas as these are the most commonly installed SuDS around buildings, where there is less available surface area.

8 RESEARCH APPROACH. INNOVANT MATERIALS ON PERVIOUS PAVEMENT

Sustainable drainage systems, as mentioned above, are relatively modern. The use of pervious pavements dates to the last two decades, but it is only in the last decade that they have been developed to a significant degree. Before talking about innovant materials, it is necessary to know the conventional materials used in pervious pavements.

As such, pervious pavements have two ways of having this characteristic, either they are a permeable material, or their disposition allows permeability:

- Permeable material: permeable material pavements are those that are composed of a porous material that allows water to pass through.
- Permeable disposition: pervious pavements are those that may be made of an impermeable material but are arranged in such a way that water is allowed to pass through to lower layers.

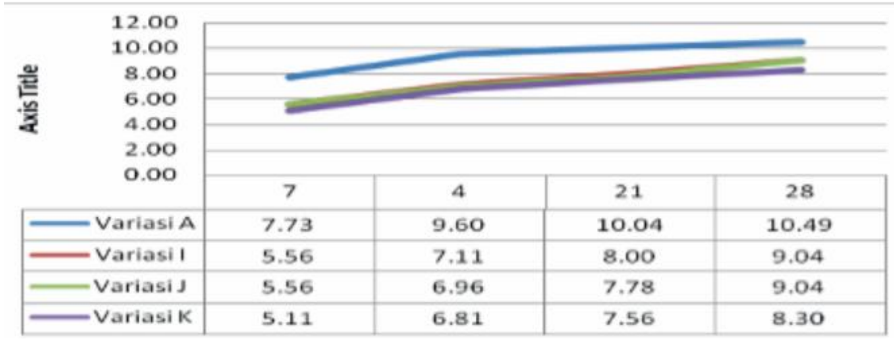
There are many variations of pervious pavements with different materials and dispositions. The most common are porous asphalt as a permeable material (PA) and concrete, either as porous concrete (PC) or as concrete blocks arranged in a permeable distribution (PBP). In this section we will go a step further, with a research approach, and we will talk about innovative materials that have not been so widely used in pervious pavements, but that several studies have shown their favorable behavior.

8.1 Ceramique tiles as coarse aggregate

Different studies have been carried out to test the possibility of using waste as a substitute for the aggregates that could be present in porous concrete (PC) or porous asphalt (PA). Among the possible wastes to be used, studies have been found that use ceramic type materials for this substitution. In order to validate the use of these materials, it is necessary to verify through tests that the losses in compressive strength expected from the use of ceramic elements are admissible and that they respond favorably to the specifications on hydraulic conductivity.

Beyond the strength due to the microstructure of the ceramic material, it is important to note that the compressive strength and density of permeable concrete or asphalt depends on the aggregate size. Larger aggregates generate lower compressive strengths.

The study on the replacement of different amounts of coarse aggregate in porous concrete pavement with roof and ceramic tile fragments tiles (Prahara, E. and Melaini, M., 2014) showed that favorable compressive strength results were achieved by replacing up to 30% of the aggregates. This type of pervious pavement achieved only a 2 MPa reduction in the compressive strength of the porous concrete. The main cause for this decrease in compressive strength is the high porosity, which at the same time decreased the density of the porous concrete. On the other hand, this high porosity allowed for better hydraulic conductivity and high water absorption.



	7	4	21	28
Variasi A	7.73	9.60	10.04	10.49
Variasi I	5.56	7.11	8.00	9.04
Variasi J	5.56	6.96	7.78	9.04
Variasi K	5.11	6.81	7.56	8.30

Figure 14. Comparison compressive strength of each variation. Source: Prahara, E. and Melaini, M., 2014

The results regarding compressive strength, while favorable for another case study, conclude that it is not possible to use this type of pavement in the project area. Generally, porous concrete should withstand compressive strengths of around 20-25 MPa, without additives. This study determined in its samples, the maximum strengths (without addition of roof and ceramic tiles aggregates) was of around 10 MPa, which can be identified in blue in the image above. This study would affirm, in the best-case scenario, that the use of these materials could match the properties of conventional porous concrete.

Another study that serves as an example to guide our research is the one carried out with the replacement of coarse aggregate with ceramic tile waste (Sathyanarayanan, V et al., 2020). The results indicate that the maximum compressive strength is obtained with the substitution of 25% of natural coarse aggregates by the ceramic tile aggregates, coinciding with the limiting percentage of the previous study and in this case higher compressive strengths are obtained, corresponding to 21.6 MPa compared to 22.8 MPa for conventional porous concrete.

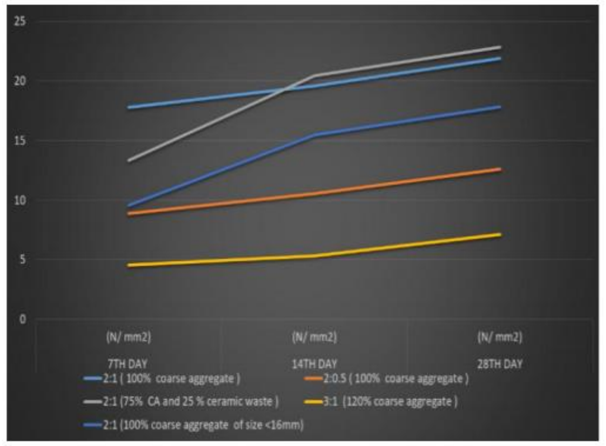


Figure 15. Results on replacement of course aggregate and their compressive strength. Source: Sathyanarayanan, V et al., 2020

As can be seen from these studies, although the addition of ceramic tiles as coarse aggregate provides a more than acceptable compressive strength compared to conventional porous concretes, the proposed mixes do not improve the mechanical properties of the pavement.

8.2 Recycled concrete as coarse aggregate

Another option proposed among researchers is the use of recycled concrete as coarse aggregate in porous concrete. A study was conducted comparing different types of conventional porous concrete pavement with recycled concrete coarse aggregate (Bhutta, A. et al., 2013). To compensate for the decrease in compressive strength due to the addition of this type of material, styrene butadiene rubber-based redispersible polymer powder and latex was added to the mix. The study showed that the void ratio was significantly increased with recycled concrete. However, the addition of the polymer powder and latex to improve the mechanical behavior decreased the void ratio of the mixture. Nevertheless, this mix had an acceptable void ratio of 22-28% and an acceptable conductivity for use as a pervious pavement in drainage systems.

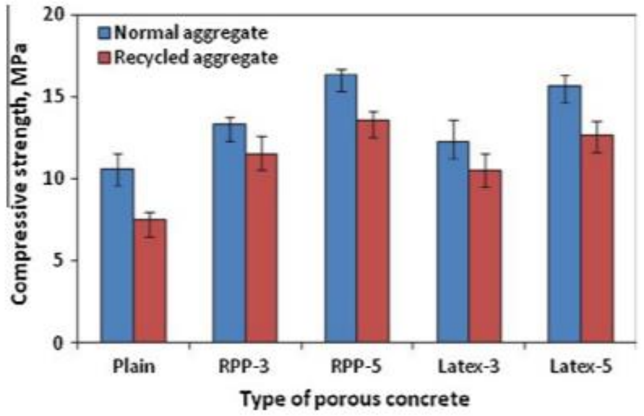


Figure 16. Comparison on compression strength of recycled concrete aggregate. Source: Bhutta, A. et al., 2013

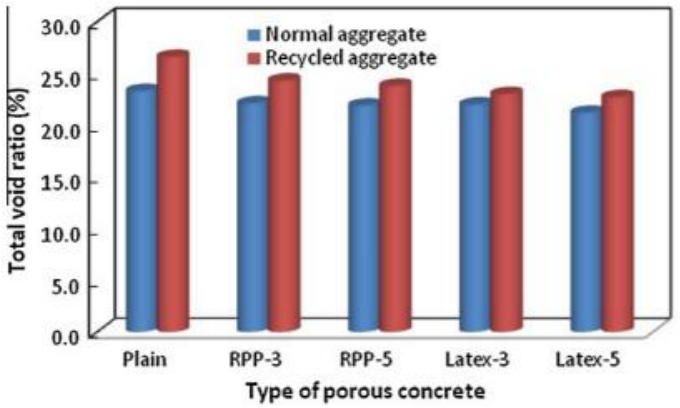


Figure 17. Comparison on void ratio of recycled concrete aggregate. Source: Bhutta, A. et al., 2013

As expected, the compressive strength decreased with recycled concrete, but the use of polymer powder and latex increased the compressive strength by up to 79%, at best 13 MPa. Finally, the study concludes that the use of recycled aggregate along with polymer modification could produce acceptable porous concrete having both sufficient drainage and strength properties.

8.3 Bio-based polyurethane binder

One option presented as an unconventional material is the use of bio-based polyurethane as binder. A recent study (Lu, Guoyang et al., 2019) has conducted trials to compare porous asphalt with bitumen with a proposed new pavement combination. The proposed combination is based on replacing the natural aggregate with recycled ceramic aggregate and the bitumen with a bio-based polyurethane binder.

After carrying out different tests, checking the resistance to different parameters of both pavements, it was determined that the pavement with recycled ceramic aggregate and bio-based polyurethane binder was superior in the following aspects:

- Compressive strength
- Resistance to permanent deformation
- Hydraulic conductivity
- Lower aggregate wear
- Increased skid resistance
- Significant reduction of energy and greenhouse gas emissions (no need to be heated).

All these characteristics mean that the proposal achieves environmental benefits, but also benefits in terms of mechanical and functional properties, improving the hydraulic performance of pervious pavements.

8.4 Ceramic tiles disposition. Life CERSuDS Project

Still with ceramic material, but with a broader vision than aggregates, research has been carried out on a project that seeks to generate a pervious pavement based on the disposition of ceramic tiles. The research into permeable paving composed of ceramic tiles comes from the LIFE program financed by the European Union. This program seeks to implement, update and develop the European Union's environmental and climate policy and legislation. This is how the Life CERSuDS program was created, which seeks to implement the use of permeable ceramic pavements, based on the use of ceramic tiles of low commercial value.

In order to verify the behavior of this type of paving, a demonstration project has been developed in a section of a street in Benicàssim, Castellón. It is no coincidence that this project was developed in Castellon. To give some context, Castellon is a region with a large ceramics industry and therefore has a lot of surpluses that cannot be used, in terms of quality, for the purpose for which they were created. Therefore, this program allows to give a second life to this excess of ceramic material.

The pervious pavement is based on a pervious pavement made up of ceramic pavers of low commercial value laid on draining bases that percolate the water into the ground, carrying the excess to a storage located under the cycle path that allows it to be recovered for irrigation of the landscaped areas and which also acts as a collector, delaying and reducing the contribution to the network during peaks of rainfall.

Figure 18. Disposition of ceramic on Benicassim, Castellon. Source: Life CERSUDS

During the development of the project, specifications were defined, designed, tested and validated for the permeable ceramic system from a performance, economic and environmental point of view, with the aim of guaranteeing the durability of the system and promoting its incorporation in urban spaces, through the development of a design guide.

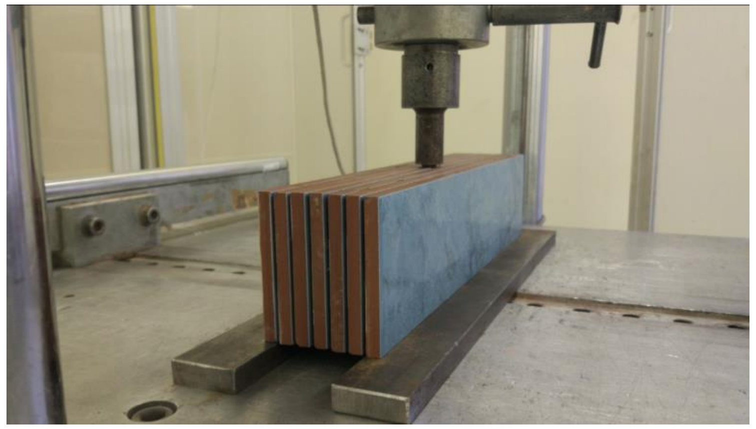


Figure 19. Compression strength test. Source: Life CERSUDS

8.5 Waste vermiculite and dolomite as mineral additives

There have been references to the use of waste materials as a substitute element. However, there are several studies that use waste material as an admixture to improve the properties of concrete. In particular, a study on the addition of dolomite and vermiculite demonstrates the improvement of strength properties (Armin, A. et al., 2018). Dolomite is a mineral that possesses high compressive strength and vermiculite is a lightweight mineral that combined with ordinary lime sand has been found to give results of improved strength properties.

The improvement of strength properties depends on the dosage. This particular study varied different dosages between 5 and 30% to analyse their behaviour. The results went from a control sample with a compressive strength of 10 MPa to a value of 24.8 MPa.

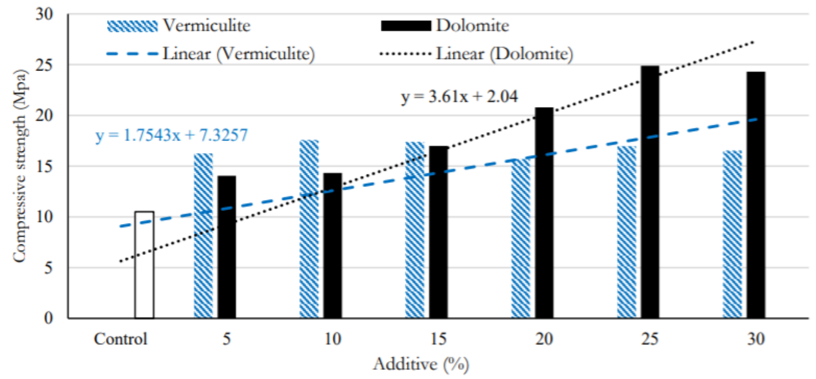


Figure 20. Compressive strength on addition of dolomite and vermiculite. Source: Armin, A. et al, 2018

An important aspect to consider in the addition of minerals is the fact that they are fine aggregates. This has the consequence that the permeability decreases with the addition of dolomite and vermiculite. As can be seen in the picture below, the addition of dolomite and vermiculite decreased the permeability from 34 cm/min to values of 1.8 cm/min.

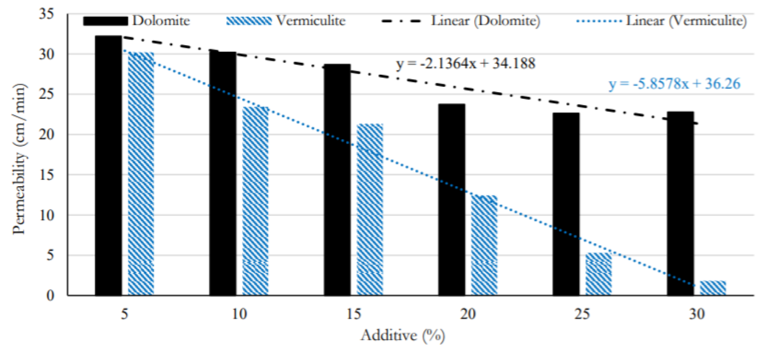


Figure 21. Permeability on addition of dolomite and vermiculite. Source: Armin, A. et al., 2018

However, as indicated above, this study included the addition of ordinary lime sand along with the vermiculite. In this way, the worst result of 1.8 cm/min was increased to 12.96 cm/min.

In conclusion, it was shown that the addition of dolomite, vermiculite and lime sand increased the compressive strength by 124%, while decreasing the permeability by 39%. In addition, the use of vermiculite reduced the weight of the concrete by 100 kg, making the pavement lighter and less dense. Overall, although a small reduction in permeability was observed, the use of waste vermiculite and dolomite significantly improved the mechanical properties of the porous concrete.

8.6 Organic polymers as additives

Among the different studies that seek to improve the properties of concrete, there is a study (Yang, J. & Guoliang, J., 2003) in which different samples and dosages of silica fume, superplasticizer and organic polymers are tested, demonstrating an improvement in the compressive strength of the pavement. Other aspects such as water penetration, abrasion resistance and durability to freezing and de-icing were also analyzed and finally obtained good results.

8.7 Suitable choice for site requirements

After carrying out a global analysis of the different possibilities of non-conventional materials for the manufacture of pervious pavements, it is shown that the use of recycled or waste materials represents a solution that benefits the life cycle of the materials, as well as a reduction in energy and pollution measured in CO2 emissions by avoiding the manufacture of new materials.

Several studies have shown that the use of materials such as ceramic tile, recycled concrete and polymer-rubbed concrete in place of course aggregates reproduce very similar strength properties to conventional porous concrete at the appropriate dosage. In many cases, not only is this the benefit, but with the optimum formulation, higher permeabilities than conventional porous concrete can be achieved, thus contributing to a better performance of the pervious pavement in the drainage system. However, although these materials reproduce similar compressive strengths to porous concrete, the industrial site studied has specific characteristics where heavy traffic is present. Therefore, one of the main requirements for the execution of the project is the compressive strength.

Mainly, the pervious pavements in the study area will be used for car parking of private vehicles, which will be discussed later. From this statement, it can be deduced that it is not necessary a higher compressive strength than a standard one. However, the continuous passage of heavy traffic in the industrial area makes it necessary to consider the resistance to eventual loads from this heavy traffic when designing the cross-section. Therefore, it has been determined that it is not appropriate to accept pervious pavement cross-sections where compressive strengths are in the order of 15-20 MPa. In the study area, higher compressive strengths should be obtained. Among all the options, the modification of porous concrete with silica fume, superplasticizer and organic polymers appears to be the most suitable for implementation in our study.

In a future study in more detail, it would be ideal to carry out tests that mix both techniques and to be able to use waste or recycled materials in the porous pavement together with additives that increase the mechanical properties, whether it is the option chosen with silica fume, superplasticizer, and organic polymers or with other additives such as dolomite and vermiculite. However, today we do not know the behavior of this type of combination of materials, let alone their optimal dosage. Therefore, for this work we are going to stick with the option of a porous concrete section with silica fume, superplasticizer and organic polymers as admixtures.

ANNEX Nº2: INITIAL DATA AND DESIGN CONDITIONS

SUSTAINABLE DRAINAGE SYSTEMS (SUDS) IN INDUSTRIAL AREAS:
APPLICATION TO THE INDUSTRIAL AREA IN QUART DE POBLET (VALENCIA)

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Summary

TABLES INDEX

FIGURES INDEX

1 OBJECT

This annex aims to present all the starting data that have been defined in order to subsequently make the different proposals of alternatives for the drainage network system and develop the optimum solution adopted.

This annex details the information, conveniently selected and treated, in order to define the relevant aspects and parameters of the study area such as urban planning, topography, geology, hydrogeology, geotechnics, pluviometry and, ultimately, a very important initial data that corresponds to one of the main objectives of the development of the work, which is the natural state of the catchments, their behavior and the objective parameters that will help to design the different solutions.

2 URBAN AND TERRITORIAL PLANNING

The urban and territorial planning of the study area is carried out by means of the General Urban Development Plan (PGOU) of Quart de Poblet, published in 2002. The following are the cut-outs to have a general visual representation of the project area.

Figure 1. PGOU Quart de Poblet cut-out. Source: Ayuntamiento Quart de Poblet, 2002

The PGOU identifies urban, non-urban and developable urban land in Quart de Poblet. Within the study area, the subcatchments for private use and the public facilities corresponding to roads and green areas are identified.

Knowing the urban and territorial planning serves as a starting point for interpreting the type of land, which plays a very important role in determining the hydrological behaviour of the study area.

Among the public facilities on the map, two green areas can be identified, one located to the east of the study area and the other located just to the centre-north. Both green areas are identified by the JL sign indicating that they are of the "garden" type. Below is a diagram of the different land uses in the study area that has been implemented from the PGOU:

Figure 2. Land uses division

Figure 3. Land uses division on photo map

For the development of the work, the study area has been divided into three main land uses, corresponding to the hydrological response of each one and the amount of runoff they generate. This classification will be discussed in detail later, but in this section we will proceed to identify them, with the land corresponding to roads in blue, green areas in green and private plots in pink.

3 TOPOGRAPHY

The representation of the topography of the study area has been obtained from two different public sources, one of which is the topographical map of the Valencian Cartographic Institute (ICV) *Instituto Cartográfico Valenciano* and the other is the topographical map of the National Geographical Institute (IGN) *Instituto Geográfico Nacional*.

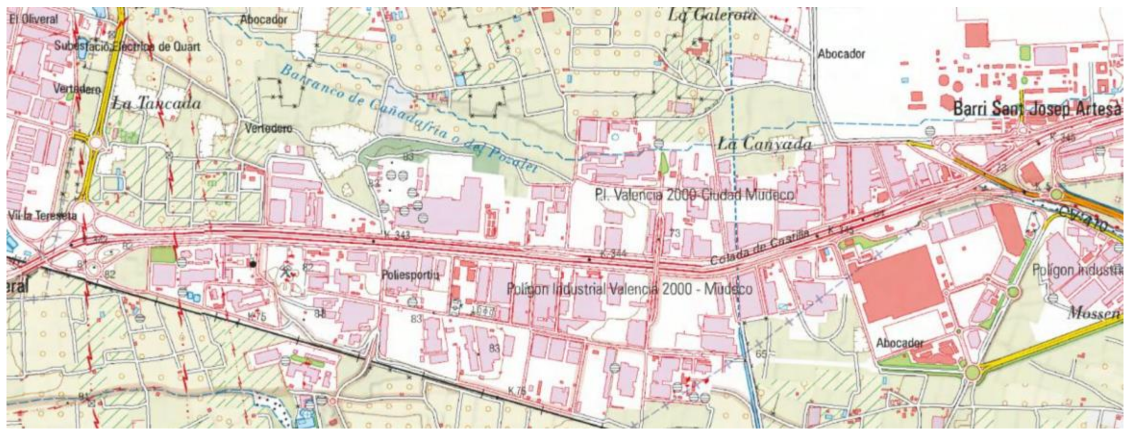

Figure 4. Topographic map from IGN for the zone of study. Source: GVA map viewer

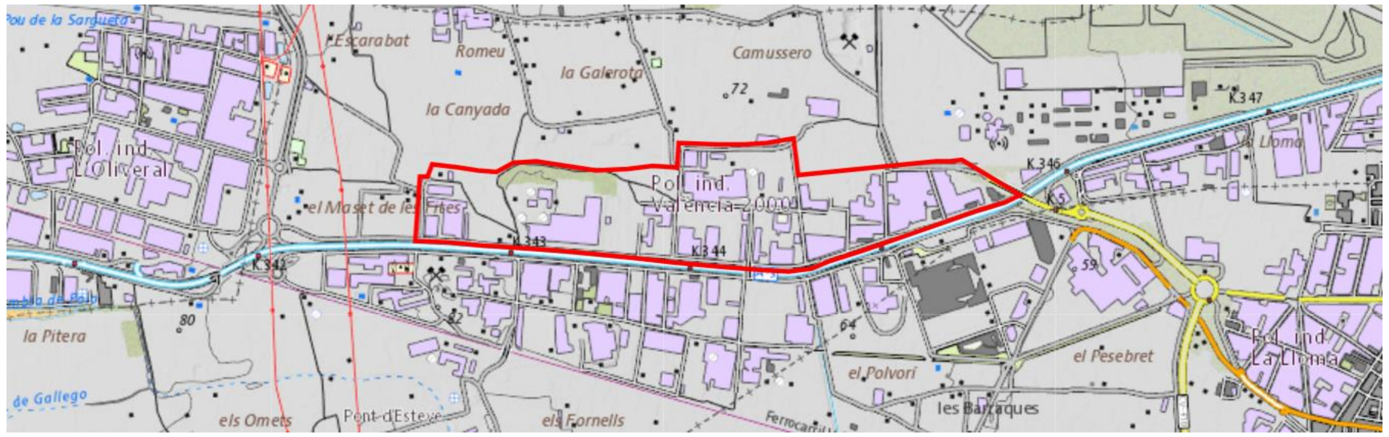

Figure 5.Topographic map from ICV for the zone of study. Source: GVA map viewer

Identified as considerably flat terrain, the overall slope is less than 2%. The level curves represented are too distant to be able to interpret the slopes of the terrain on the map. However, it was possible to obtain the coordinates of any point on the map in the ICV online map viewer. In this way, the necessary coordinates have been obtained to define the slopes of the elements that make up the layout of the industrial estate, which are important when designing and dimensioning the rainwater drainage system, with the intention that the whole system should work by gravity, without the need for pumping or infrastructures that make the solution more expensive.

The determination of the slopes of the land is important in order to determine the natural course of surface runoff and thus know the volume of water to be treated on each surface and better design the drainage system. In addition, slopes are a determining factor, since certain sustainable drainage systems are limited by the slope of the land. *Annex Nº2: Initial data and design parameters* contains an analysis of the slopes accepted by the SUDS and the limitations they place on the design of the system.

The analysis of private residential subcatchments is not included in the object of the study, although it is taken into account for the generation of runoff. Therefore, the direction of the slopes will be determined following a coherence with the cartographic data and always in accordance with a possible future SUDS action in the private subcatchments.

The following image shows the plan of the longitudinal slopes of each street subcatchment obtained after processing the information from the cartographic maps. This information can also be found to scale in *Document Nº2: Plans.*


Figure 6. Longitudinal slopes on catchment N1

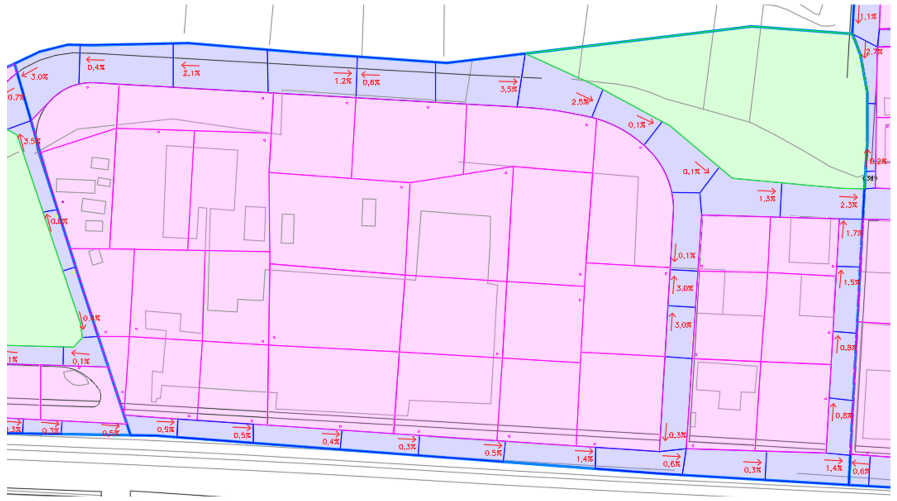
Figure 7. Longitudinal slopes on catchment N2

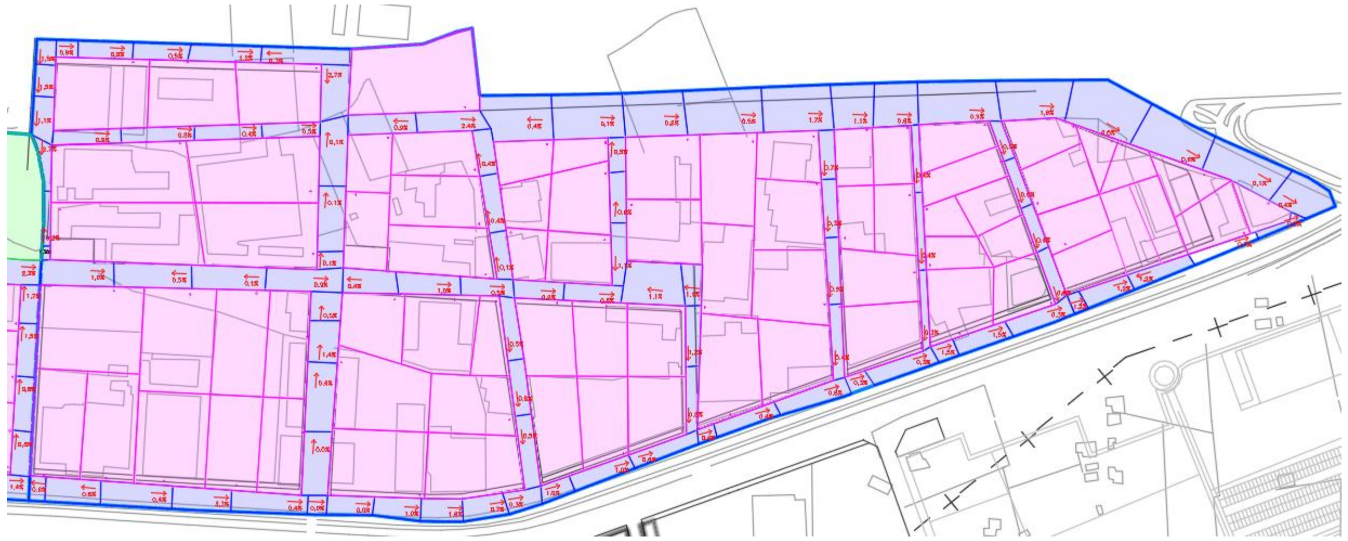
Figure 8. Longitudinal slopes on catchment N3

4 GEOLOGY, HYDROGEOLOGY AND GEOTECHNICS

The geological study is based in part on information gathered from the Spanish Geological and Mining Institute (IGME) *Instituto Geológico y Minero de España*, specifically from the report on " Sheet 722 (29-28): Valencia ", from which most of the data on the characteristics of the terrain have been obtained.

The maps consulted are as follows:

- Sheet Nº 722 (Valencia) from the Geological Map of Spain. Series Magna. Scale 1/50.000, IGME, year 1974.
- Development plan. Flood risk zoning. Sheet 722. Scale 1/80.000, Plan de Acción Territorial sobre Prevención del Riesgo de Inundación en la Comunitat Valenciana. year 2015.
- Development plan. Flood hazard zoning. Sheet 722. Scale 1/80.000, Plan de Acción Territorial sobre Prevención del Riesgo de Inundación en la Comunitat Valenciana. year 2015.
- Soil permeability and aquifer recharge - Critical areas in the face of climate change. Sheet 15, l'Horta. Scale 1/50.000. Instituto Cartográfico Valenciano. year 2020.

The following is a general description of the geological characteristics of the area that must be known to carry out the study of solutions to meet the objectives of this work.

4.1 Geological context

The study area, which belongs to the synclinal axis of the Cadena Ibérica, was formed during the Miocene as a consequence of the shortening of the area occupied by the Mesozoic sea, which produced the emersion of areas of positive relief and depressed areas aligned along Iberian axes.

The Valencian region is located in one of these depressions, which was invaded by the sea in the Lower Tertiary period. This sea deposited sandy materials, initially conglomerates, forming a gulf of larger dimensions than the present one. In the Upper Tertiary, the sea regressed, creating a lagoon-like area, in which marl and limestone were deposited, which slowly filled in due to the contributions of water flows from the surrounding areas.

Subsequently, the generation of a more extreme climate than the present one and the hydrological system gave rise to quaternary deposits from the erosion of the mountains upstream, while the coastline retreated, abandoning the old littoral sediments.

4.2 Stratigraphy

Sheet 722 is located in the province of Valencia and is mostly occupied by terrain from the Quaternary period. There is a part of tertiary soils from the Neogene age and, to a small extent, Cretaceous period soils can also be observed.

The study area is located in the soils of the **Quaternary period**, specifically in the **continental deposits** corresponding to the **Lower Pleistocene with calcareous crust** and to the **Upper Pleistocene** with **modern stream mantles**, being red sandy clays with crustal edges.

To know a bit more about the study terrain, as well as its surroundings, the type of stratigraphy of the sheet has been detailed, with more emphasis on the study area.

4.2.1 Quaternary

The Quaternary is of special interest within the Valencia Sheet due to its large surface area, as well as the variety of its formations. As a whole, it is presented as an extensive pre-littoral plain, occupied for the most part by the Albufera and associated sediments and by the flood silts of the River Turia. Three distinct types of deposits have been distinguished, with different variants within each of them. Continental deposits, marine deposits and mixed deposits.

Continental deposits

- **Crust:** Formed by zoned limestone, it appears overlying the Upper Miocene limestones in the area around Picasent. It is made up of white and pinkish levels, alternating with very variable strength from one point to another. The most interesting particularity of this crust is that it includes subfossil gastropod shells which have been classified as Iberus alonensis, which is very abundant. The complete absence of Leucochroa candidissina, which always accompanies Iberus alonensis in the Holocene sediments of the region, seems to suggest that these deposits are older. The stratigraphic position with respect to the other crusts that appear in the area allows us to place it, in the same way, in an ancient Quaternary. Its genesis is due to the remobilisation of the carbonates of the Miocene limestones by water mantles on gentle slopes. This intermittent flow would allow the hardening of the crustal layers due to their temporary exposure to the air.

- **Modern stream mantles:** These form a discontinuous border that sometimes overlies the older stream mantles, which are smaller in extent. The fundamental difference is that they are not crusted in any way. The deposit is made up of red clays, with levels of sub-rounded edges. Its genesis is similar, although chronologically later, to that of the former.

4.3 Tectonics

Almost the whole of the Valencia sheet is located in a wide morphological depression of complex tectonic origin. This depression, geographically known as the Huerta de Valencia, extends between the sea and the foothills of the Chiva and Buñol reliefs and is flanked to the north by the Náquera reliefs, and to the south by the Cullera-Alginet reliefs.

The depressed area represents a synclinal axis of the Cadena Ibérica, in which materials have been deposited clearly after the main moment of compression which created the "Iberian" structures. On the other hand, this tectonic depression has been affected by later movements causally related to the tectonics of the "Betic" areas located further south, which in turn must be responsible for the more recent movements of the Mediterranean coasts.

From a structural point of view, the emerging materials on this sheet can be divided into two groups:

- Upper Cretaceous materials in their terminal part, folded in a broad style.
- Materials from the Upper Tertiary and Quaternary, not deformed.

4.4 Hydrogeology

4.4.1 Flood hazard

The flood hazard map has been provided by the Territorial Action Plan on Flood Risk Prevention in the Comunidad Valenciana (PATRICOVA):

- Development plan. Flood hazard zoning. Sheet 722. Scale 1/80.000, Territorial Action Plan on Flood Risk Prevention in the Comunidad Valenciana. Year 2015.

The development of the PATRICOVA plan has been based on the contents of the Thematic Cartography on "Delimitation of Flood Risk at Regional Scale in the Valencian Community" published by the COPUT (*Conselleria d'Obres Públiques, Urbanisme i Transport*) in 1997, as well as previous PATRICOVA plans.

In this way, PATRICOVA has developed a methodology for estimating hazard, classifying them in levels ranging from 1 to 6, and has also identified areas of geomorphological hazard.

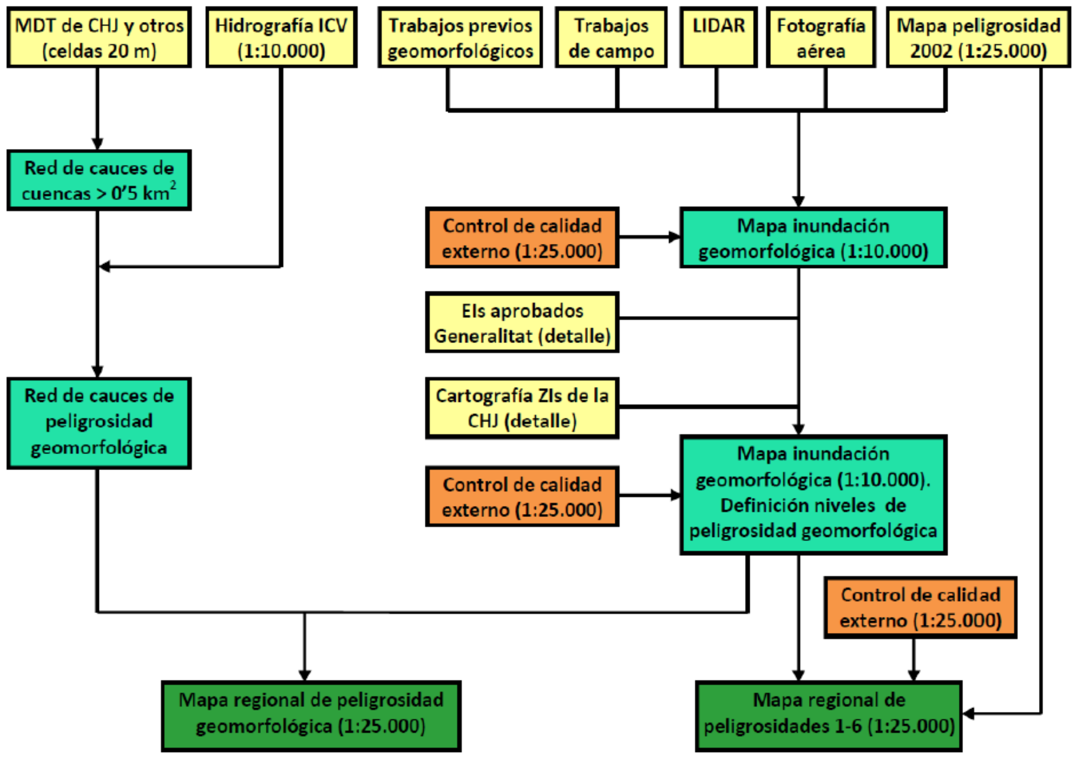

Figure 9. Diagram of the general procedure for the determination of flood hazards in PATRICOVA. Source: IIAMA

Flood hazard levels are classified according to two parameters: on the one hand, the frequency of a given flood, which is represented by the return period and, on the other hand, the magnitude of the flood, which is represented by the maximum level or depth reached by the water. In this section a cut-out provided by the GVA map viewer is given:

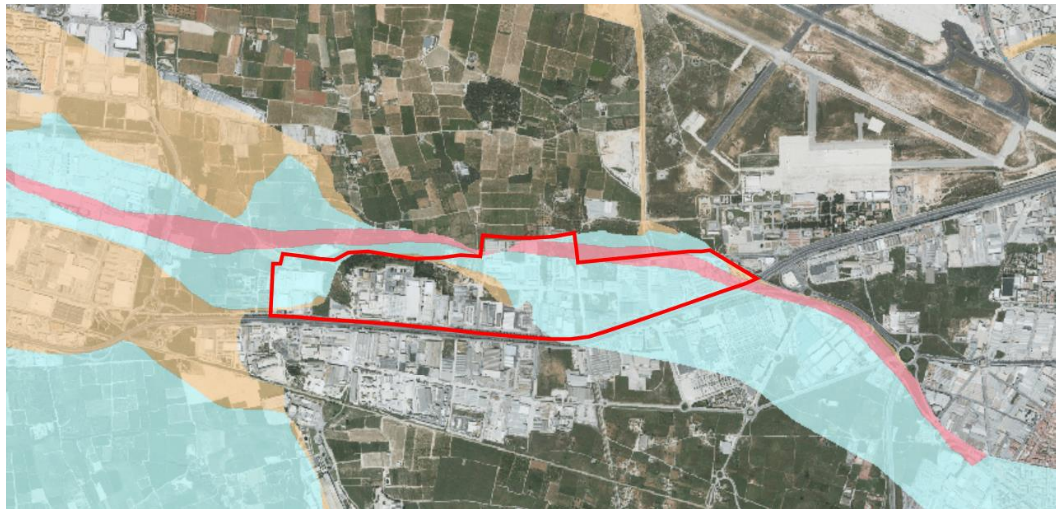

Figure 10. PATRICOVA flood hazard map. Source: GVA map viewer

The northern part of the industrial estate is considered a **level 1 hazard** zone, identified in pink on the map, with a high return period frequency of 25-year return period and a depth exceeding 0.8 meters. The whole of the N3 catchment area and a large part of the N2 is classified as a **level 3 hazard** zone, identified in light blue, with a high frequency of 25-year return period and a low depth of less than 0.8 meters.

In terms of geomorphological hazard, identified in orange, the study area is characterized by a small **geomorphological hazard** zone to the east and in the center of the industrial estate.

4.4.2 Flood risk

The flood risk assessment carried out by PATRICOVA has been based on Directive 2007/60/EC and Real Decreto 903/2010 on vulnerability to flooding. In this way, these regulations have established minimum contents to be considered in order to determine the risk to which a territory is subject due to flooding.

The contents are based, fundamentally, on the incorporation of three factors that could be damaged because of a flood:

- Economic factors
- Social factors
- Environmental factors

To quantify it, flood risk is measured in units of damage per floodable area.

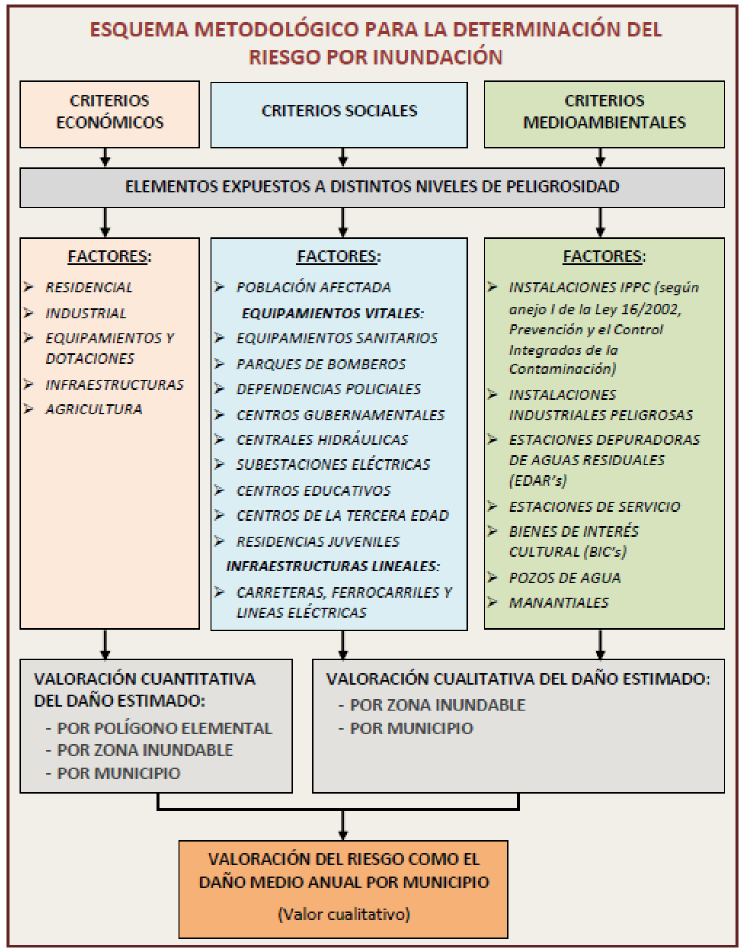

Figure 11. Methodological scheme for determining vulnerability and risk as an assessment of flood damage. Source: Generalitat, de ordenación del territorio, urbanismo y paisaje, de la Comunitat Calenciana, 2015.

The map provided by the Territorial Action Plan on Flood Risk Prevention in the Valencian Community (PATRICOVA, 2015) shows us on sheet 722 the flood risk zoning:

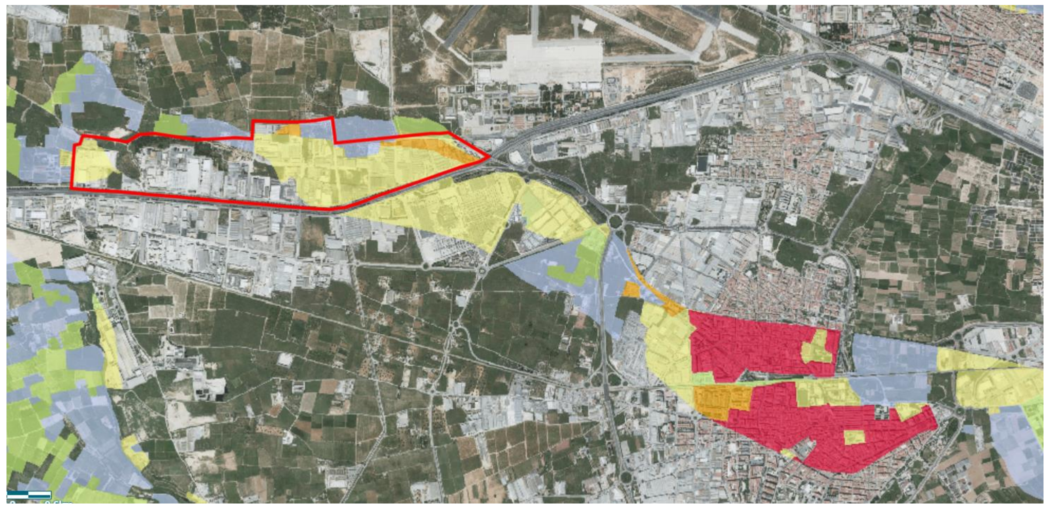

Figure 12. PATRICOVA flood risk map. Source: GVA map viewer

The flood risk map shows that the project area is an area where the risk is generally **medium**, in yellow, containing some areas of **low** risk, in green, and **very low** risk, in blue, and, to a lesser extent, areas of **high** risk, in orange. As indicated in the problem of runoff water discharges, the area close to the industrial estate is the municipality of Aldaia with a very high risk of flooding, identified in red.

4.4.3 Permeability

The permeability of the soil has been analysed on the basis of the map provided by the Valencian Cartographic Institute (ICV), which classifies the entire area of the Comunidad Valenciana from very low to very high permeability.

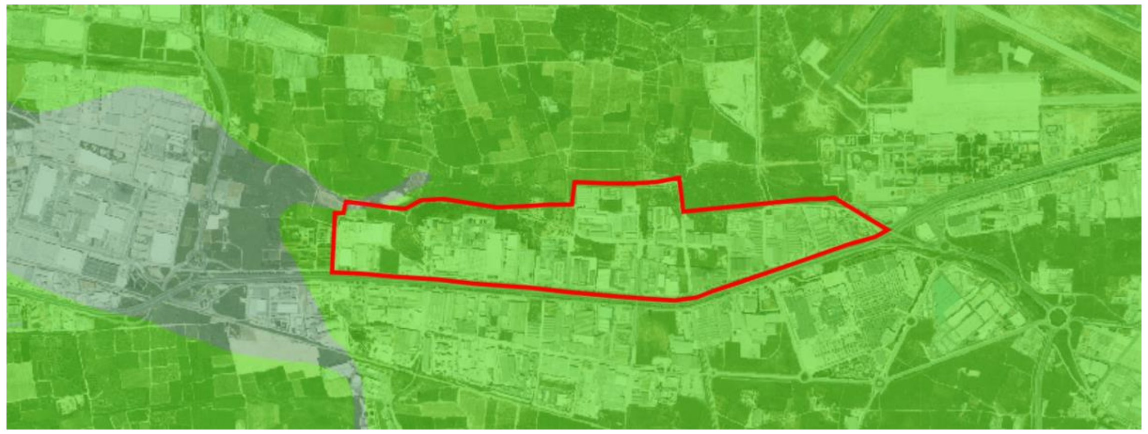

Figure 13. Map of permeabilities. Source: GVA map viewer

The entire project area is classified as a **high permeability** zone in light green, being bordered to the east by a very high permeability zone in dark green. Unfortunately, the ICV does not provide a value or a range of values in which the permeability coefficient of the study area could be included. A search for further references leads to the conclusion that no other agency provides this information.

Following DIN 18130-1: 1998 [15], it has been possible to obtain a range of permeability coefficient values based on their classification:

Table 1. Permeability values based on the classification. Source: Norme DIN 18130-1, 1998

Permeability	Value (m/s)
Very high	$>10^{-2}$
High	10^{-1} a 10^{-4}
Moderated	10^{-4} a 10^{-6}
Low	10^{-6} a 10^{-8}
Very low	$<10^{-8}$

Due to the lack of information regarding the exact value of the permeability coefficient, it has been considered that the permeability of the project area is similar to the permeability of the "Moli d'Animeta" sector in the municipality of Quart de Poblet, for which specific information on permeability has been available thanks to the tests carried out between December 2018 and February 2019 by the company Green Blue Management (Martínez, L., 2019).

The tests carried out were trench permeability test campaigns in which a minimum value of permeability coefficient corresponding to 19.3 mm/h, $5.34 \cdot 10^{-6}$ m/s, was obtained.

The similarity of the geology and geomorphology of the study area compared to this sector shows that it is acceptable to consider the same permeability. Therefore, it has been determined that the project area can be considered with the same permeability, or even higher than that obtained in the urbanisation, but that the permeability obtained in the "Molí d'Animeta" sector will be kept as it is more conservative.

Following the recommendations of The SuDS Manual (Woods Ballard et al., 2015), a safety factor must be determined to be applied to the permeability for the hydraulic design of infiltration systems. It has been decided to use a safety coefficient of 1.5, as the lamination-infiltration structures to be designed are to manage frequent rainfall events and the consequences of failure of these systems have been considered low.

Size of area to be drained	Consequences of failure		
	No damage or inconvenience	Minor damage to external areas or inconvenience (eg surface water on car parking)	Damage to buildings or structures, or major inconvenience (eg flooding of roads)
< 100 m²	1.5	2	10
100–1000 m²	1.5	3	10
> 1000 m²	1.5	5	10

Figure 14. Suggested factors of safety for infiltration systems. Source: Woods Ballard et al., 2015

Following the above, the coefficient of permeability of the soil for the design phase is determined to be:

$$Kc = \mathbf{12,9\ mm/h}$$

4.5 Return Period

Flood risk prevention and management in the Comunidad Valenciana are included in the Territorial Action Plan on Flood Risk Prevention of the Comunidad Valenciana (PATRICOVA). Since the municipality in the study area does not have specific regulations on drainage systems, it was considered that the choice of the return period for the design rainfall should be made based on this document or of the Regulations for urban drainage and sewerage works in the city of Valencia (Ayuntamiento de Valencia, 2015).

The Valencian regulations propose a design return period of 25 years for the drainage systems of the city of Valencia, slightly longer than that of the PATRICOVA. However, the Territorial Action Plan indicates that stormwater drainage in urban areas of more than 20 hectares will be designed with a level of protection of at least 15 years return period.

$$\boldsymbol{Tr = 15\ years}$$

The reason for choosing the 15-year return period indicated in the PATRICOVA is related to the application of the drainage system. Drainage systems in urban catchments cope with short, high intensity and high flow, but relatively short rainfall events. This is because the drainage surfaces are small, impermeable and with very short concentration times. It is therefore recommended that the return period should be at least 15 years. Not only that, but it is not recommended to design for return periods longer than 25 years because the events in the area generate a relatively short peak flow that can be solved by temporary storage in the roadways. Another important point that supports this recommendation is the fact that the design of drainage systems for longer return periods would be too costly in comparison with the benefits in terms of functionality and efficiency.

5 PLUVIOMETRIC ANALYSIS

The rainfall analysis has been carried out with the data obtained from the Valencia - Airport rainfall station, code 8414 A. The station is in an area very close to the area of action, at Manises airport. The data of the historical rainfall series goes from 1966 to 2019, which means a collection of data for 54 years.

The present pluviometric analysis focuses on carrying out a volumetric frequency analysis and an extreme analysis of the maximum daily rainfall for different return periods.

Of the data obtained by the pluviometric station, all those greater than 1 mm have been used, since, for the drainage system, the effects of rainfall with values of less than 1 mm are negligible.

5.1 Volumetric frequency analysis

From the data obtained by the rainfall station, the average monthly rainfall and the average number of rainy days per month for the available series have been calculated.

Table 2. Average monthly rainfall and average number of rainy days

Month	Rainfall (mm)	Rainy days
1	33,15	3
2	29,76	3
3	35,29	4
4	27,23	5
5	34,35	4
6	20,99	3
7	10,33	1
8	14,91	2
9	55,21	4
10	70,09	4
11	50,20	4
12	44,13	4

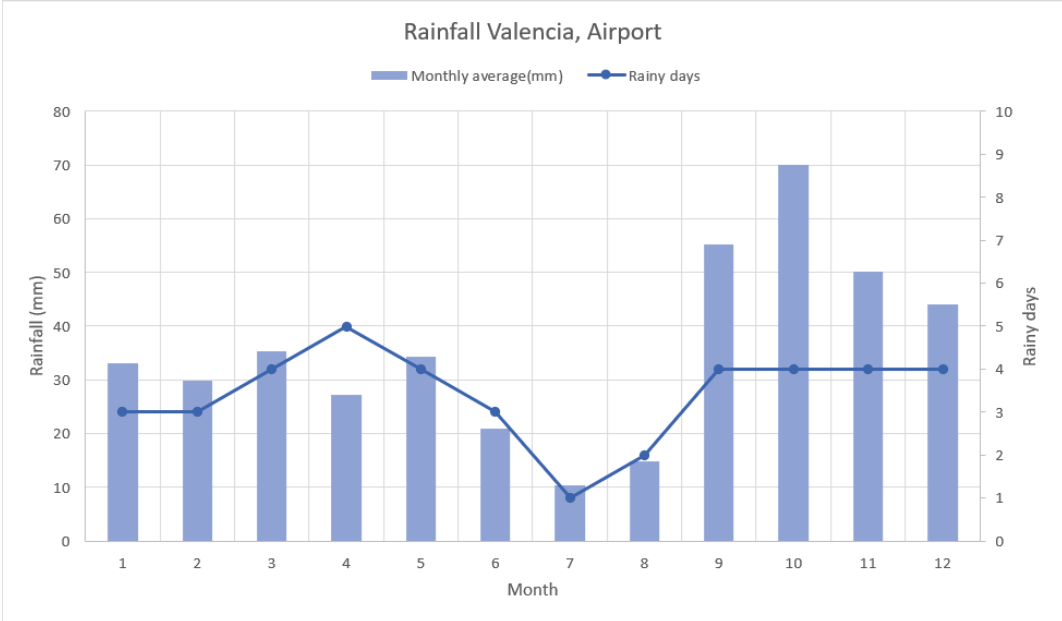

Figure 15. Diagram on average monthly rainfall and average number of rainy days

With a total of 2,270 rainy days for the recorded series, the number of days exceeding certain thresholds is obtained:

Table 3. Number of days exceeding certain thresholds

Rainfall depth (mm)	Days
15	425
30	165
60	41
90	15
120	6
150	4
180	2
210	0

With these data, we can calculate the daily rainfall volumes corresponding to the main percentiles. The calculation of these percentiles is important because the design of the sustainable drainage system will depend on them, and they are used to define the runoff reduction and treatment objectives of the system.

Table 4. Rainfall percentiles

Rainfall depth (mm)	Days	Exceedance probability	Non-exceedance probability
0	2270	100,00%	0,00%
5	1115	49,12%	50,88%
10	654	28,81%	71,19%
14	455	20,04%	79,96%
15	425	18,72%	81,28%
16	397	17,49%	82,51%
17	379	16,70%	83,30%
18	351	15,46%	84,54%
19	330	14,54%	85,46%
20	302	13,30%	86,70%
24	231	10,18%	89,82%
25	221	9,74%	90,26%
30	165	7,27%	92,73%
40	107	4,71%	95,29%
50	66	2,91%	97,09%
60	41	1,81%	98,19%
70	29	1,28%	98,72%
75	26	1,15%	98,85%
80	22	0,97%	99,03%
90	15	0,66%	99,34%
100	10	0,44%	99,56%
120	6	0,26%	99,74%
135	4	0,18%	99,82%
150	4	0,18%	99,82%
188,9	0	0,00%	100,00%

This result allows us to represent the distribution function of the daily sample rainfall, so the probability of not exceeding for each volume of daily rainfall.

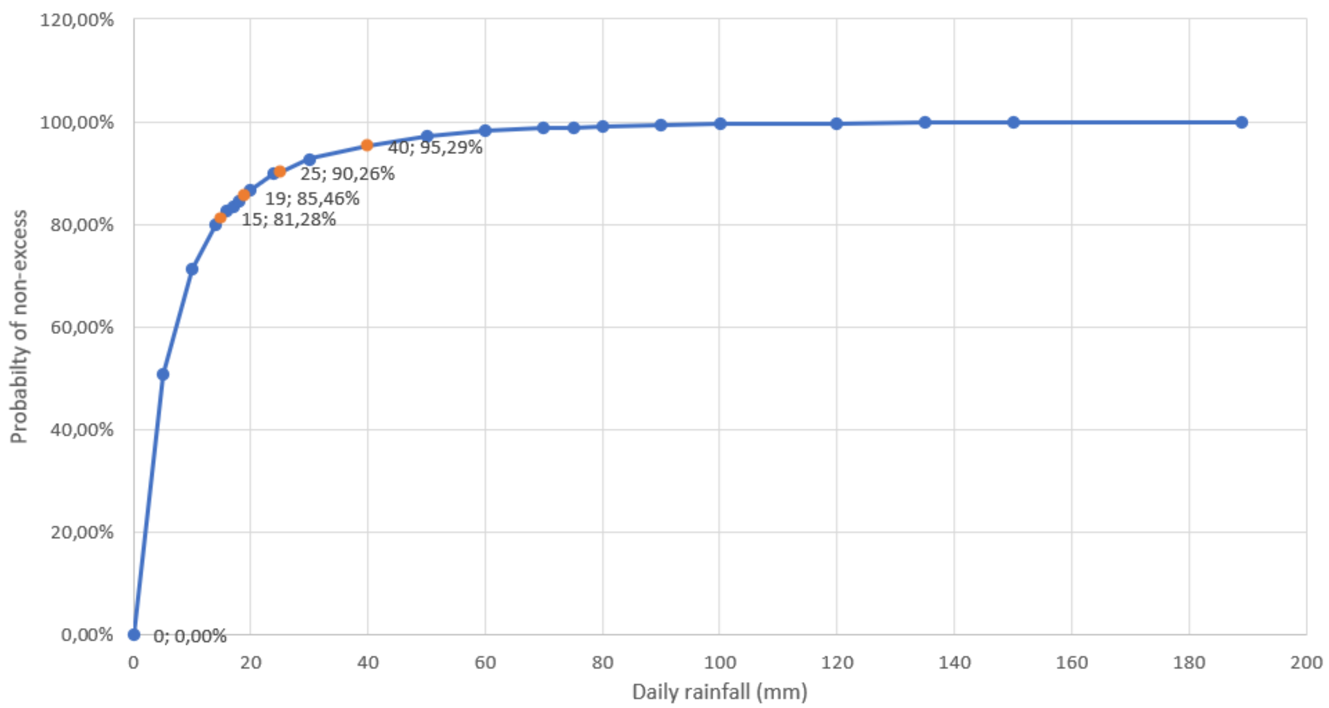

Figure 16. Graphic representation on probability of non-exceedance daily rainfall

From these results we can analyse:
- On 80% of rainy days, rainfall does not exceed 15 mm.
- On 85% of rainy days, rainfall does not exceed 19 mm.
- On 90% of the rainy days, rainfall does not exceed 25 mm.
- On 95% of rainy days, rainfall does not exceed 40 mm.

5.2 Extreme analysis

The determination of extreme events is necessary in order to be able to design a drainage system that can function satisfactorily without causing local flooding that could lead to material and environmental damage. For this reason, the highest rainfall value was determined for each recorded year:

Table 5. Maximum dairy rainfall by year

Year	P,max (mm)	Year	P,max (mm)	Year	P,max (mm)
1966	115,8	**1984**	45,5	**2002**	58,9
1967	33,1	**1985**	47	**2003**	40,4
1968	72,4	**1986**	119,9	**2004**	61,1
1969	121,3	**1987**	71,6	**2005**	43,1
1970	31,6	**1988**	69,2	**2006**	77,8
1971	186,9	**1989**	175,9	**2007**	94,2
1972	37,4	**1990**	123	**2008**	70
1973	67,8	**1991**	90,9	**2009**	85,4
1974	77,6	**1992**	55,5	**2010**	35,5
1975	77,2	**1993**	49,8	**2011**	47,3
1976	31,7	**1994**	46,6	**2012**	188,9
1977	66,6	**1995**	81,7	**2013**	43,9
1978	20,7	**1996**	48,6	**2014**	43,1
1979	33,7	**1997**	29,4	**2015**	55,4
1980	84,7	**1998**	70	**2016**	112,1
1981	27,6	**1999**	37,4	**2017**	88,8
1982	82,8	**2000**	153,3	**2018**	94,5
1983	50,3	**2001**	97,2	**2019**	49,4

These data have the following graphical form:

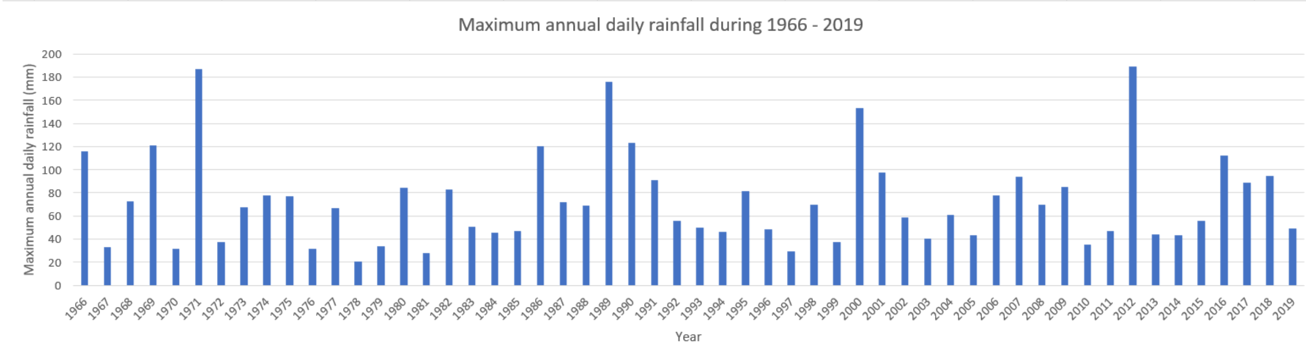

Figure 17. Maximum daily rainfall during 1966 – 2019 period

5.2.1 Statistical analysis of extreme values

Once the extreme events in the study area are known, a statistical analysis of these extreme events is carried out. The objective of the analysis of annual maximum daily rainfall is to adjust a function distribution of extremes that represents the rainfall series to obtain different quantiles for different return periods.

For the adjustment to extreme distributions, it is necessary to know the statistical parameters of the series:

Table 6. Statistical parameters of the analyzed series

Parameter	Value
Median	72,62
Variance	1572,95
Standard deviation	39,66
Asymmetric coefficient	1,34
Variation coefficient	0,55
Kurtosis	4,27

To determine the theoretical distribution that best fits the data series analyzed, the XLSTAT Excel add-in is used. This add-in makes it possible to obtain the parameters of each distribution that best represent the series provided by means of the maximum likelihood method.

The distributions that have been chosen to test their fit are those most used in the hydrological field:
- Gumbel
- GEV (General Extreme Value)
- TCEV (Two Component Extreme Value)
- SQRT-et-MAX

5.2.1.1 Gumbel

The Gumbel function, known as extreme value distribution type 1 (EV1), is the most commonly used in the study of floods, due to its easy application and the few parameters needed for its definition. It is a function that gives very accurate results at low return periods, i.e. high probabilities, but in the case of high return periods it usually gives results below those observed.

The Gumbel distribution is of the double exponential type, and its expression is as follows:

$$F_x(x) = e^{-\lambda * e^{-\theta x}}$$

where:
- λ is the shape parameter
- θ is the scale parameter

The representative Gumbel distribution values for the analyzed data series are as follows:

Table 7. Parameters of Gumbel distribution for observed series

Parameter	Value
λ	7,47853
θ	0,03653

5.2.1.2 GEV

The GEV distribution, General Extreme Value, is a generalisation of the Gumbel distribution with one more parameter added. As a three-parameter function, it reproduces extraordinary events more faithfully than the Gumbel function.

The GEV distribution has as its expression:

$$F_x(x) = e^{(-(1-\beta * \frac{x-x_0}{\alpha})^{\frac{1}{\beta}}}$$

where:
- X_0 is the location parameter
- β is the shape parameter
- α is the scale parameter

The representative GEV distribution values for the analyzed data series are as follows:

Table 8. Parameters of GEV distribution for observed series

Parameter	Value
X_0	51,7463
β	-0,24423
α	24,1957

5.2.1.3 TCEV

The TCEV distribution, Two Components Extreme Value, is a distribution that is very well adapted to the representation of series of annual maximums, because it reproduces what is known as the "dog-leg effect" characteristic of hydrological events, taking into account that there are two populations of events. One part of the events is called ordinary events of higher frequency and lower magnitude, and the other is called extreme events of lower frequency and higher magnitude.

Unlike the previous functions, which are not able to reproduce these two events, the TCEV function assumes that the ordinary and extraordinary events are derived from independent Gumbel functions, so that the maximum annual event will be the maximum of these two, its distribution function being the product of the original distribution functions.

Its expression is as follows:

$$F_x(x) = e^{(-\lambda 1 * e^{-\theta 1 x} - \lambda 2 * e^{-\theta 2 x})}$$

where:

- λ1 is the shape parameter of the ordinary events
- θ1 is the scale parameter of the ordinary events
- λ2 is the shape parameter of extraordinary events
- θ2 is the scale parameter of the extraordinary events

The representative TCEV distribution parameters for the analyzed data series are as follows:

Table 9. Parameters of TCEV distribution for observed series

Parameter	Value
λ1	7,59405
θ1	0,04076
λ2	0,46076
θ2	0,01753

5.2.1.4 SQRT-et-MAX

The last distribution analysed is the SQRT-et-MAX. It is based on the SQRT-k function and is the one used by the Ministerio de Fomento for the statistical modelling of maximum daily rainfall.

This distribution takes into account, like the TCEV, the existence of two types of ordinary and extraordinary events. However, it uses two parameters for its definition instead of four.

Its expression is as follows:

$$F_x(x) = e^{(-k*(1+\sqrt{\alpha x})*e^{(-\sqrt{\alpha x})})}$$

Donde:
- k is the scale parameter
- α is the frequency parameter

The parameters of the representative SQRT-et-MAX distribution for the analyzed data series are as follows:

Table 10. Parameters of SQRT-et-MAX distribution for observed series

Parámetro	Valor
k	26,2913
α	0,49285

5.2.1.5 Comparison of distributions and results

The analysis with the XLSTATISTICS add-in has allowed us to obtain the parameters of the distribution functions fitted to the empirical series using the maximum likelihood method.

The maximum likelihood method consists in selecting the parameters that maximise the likelihood function, which is any function proportional to the combined probability density function of all the random variables involved.

Table 11. Maximum likelihood method values

Distribution	Maximum likelihood
Gumbel	-251,918
GEV	-250,024
TCEV	-251,194
SQRT-et-MAX	-249,889

Thus, it has been possible to estimate which of the four distributions is closest to the empirical historical series, this being the **SQRT-et-MAX distribution**. Furthermore, it has been possible to verify by means of the least squares difference method that this is the one that most faithfully represents the maximum rainfall.

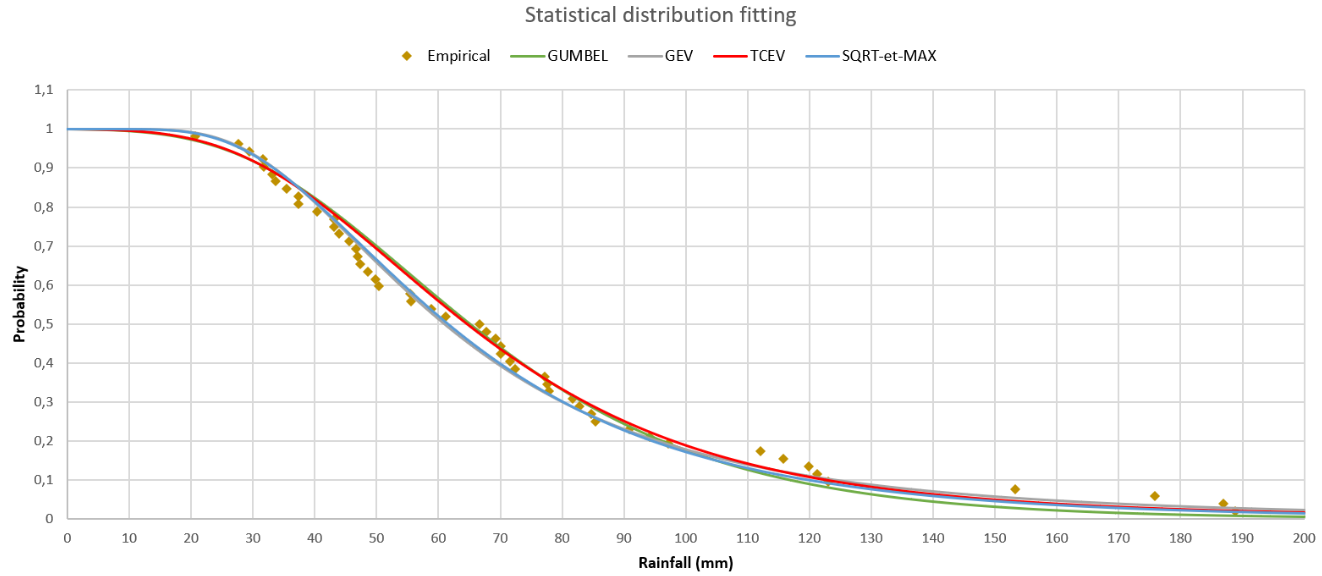

Figure 18. Representation of theorical distributions and empirical series

Once the distribution function to be used has been determined, the maximum precipitation value can be known for any return period to be studied, since the return period is defined as the inverse of the probability of occurrence.

Table 12. Maximum daily rainfall for each return period

Return period (years)	Maximum daily rainfall (mm)
2	61,60
5	94,56
10	119,70
15	135,02
25	155,15
50	184,08

The project will be designed for a return period of 15 years, so the maximum daily rainfall for the project design is:

$$Pd, \text{max} = \mathbf{135,02\ mm}$$

5.3 Design storms

Design storm surges are essential to evaluate the response of the analysed system in accordance with the desired objectives. Hydrological models of maximum rainfall are not enough to be able to model the behaviour of an event, as it is necessary to know the evolution of rainfall. For this reason, design storms are used as input for hydrological modelling.

The most used representation of a design storms is through hyetograms, which represent the intensity of rainfall as a function of time. The hyetograms are mainly obtained by two methods: with IDF (Intensity-Duration-Frequency) curves and with observed rainfall records. There is a third way for their determination, simulating stochastic rainfall models, but they have a more complex application.

In the case study the IDF curves will be used for the dimensioning of the collector network while the observed rainfall records will be used for design verification by modelling. The main reason is to make up for the weaknesses of both methods and in this way to verify the design for both, complementing each other.

5.3.1 IDF curves

The IDF curves estimate the maximum rainfall intensity corresponding to the time of concentration of the catchment for a given return period.

For the representation of hyetograms, the most used method of representing design storms is the alternating block method. This method aims to always respect the most unfavorable rainfall intensity for the IDF curve. With this method, the average intensity of a set of blocks is assumed to be equal to the intensity obtained by the IDF curve for any time interval.

For a return period of 15 years, the IDF curve obtained is as follows:

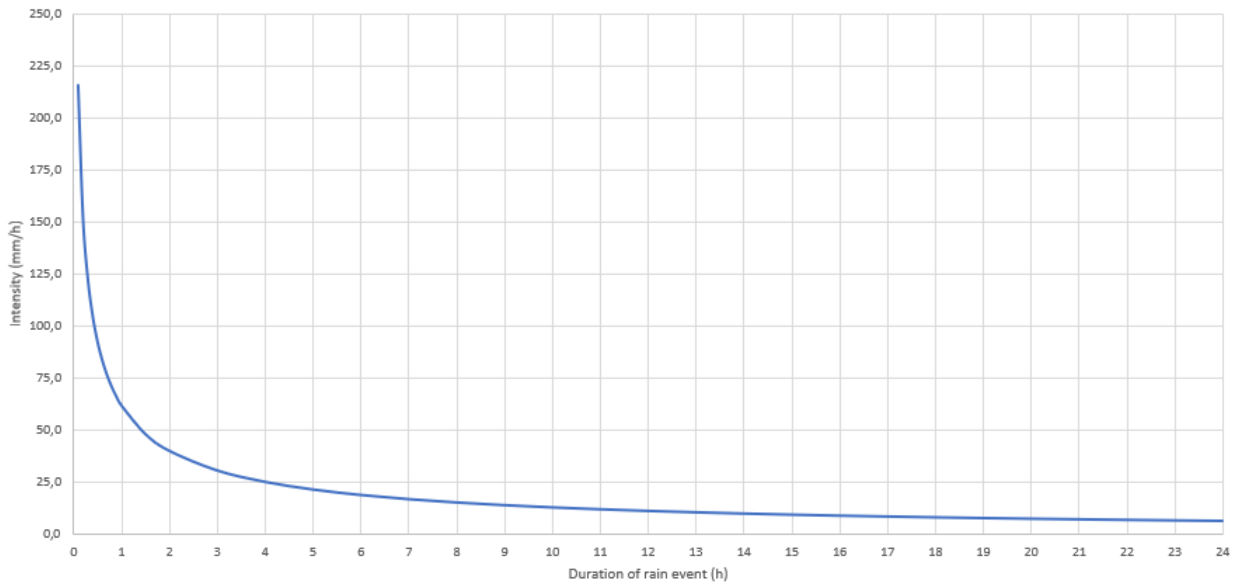

Figure 19. IDF curve for 15 years return period

The IDF curves estimate the maximum rainfall intensity. It is obtained by the modified Témez method. The reason for using this method, as well as its explanation, is detailed below in the section "Rainfall-runoff transformation model".

A negative aspect of using the IDF curves in the hydrological analysis is that it does not represent any internal structure of the rainfall. Furthermore, since the storm duration is considered randomly, the total rainfall volume is also random, so it cannot be compared with reality. It has even been found that the design storm patterns obtained with this method have a longer return period than the one used for the IDF curve from which it is derived.

It is important that the design storms calculated respect the observations of the temporal evolution of the intensities, but also the volume of rainfall since both are very conditioning parameters when designing the drainage system. This point is emphasized because volumetric analysis is crucial for the design of the temporary storage volume in SuDS.

5.3.2 Observed rainfall records: G2P method

Due to the shortcomings mentioned above, it is preferable to use other design shower methods than the one obtained by the IDF curve as they are based only on the mean maximum rainfall intensity. These are design storms that have been developed to be characteristic of the particular study area by means of internal temporal patterns observed in rainfall records.

At the IIAMA (Institute of Water and Environmental Engineering) linked with the UPV, recent studies have been carried out that seek to represent design sotrms that resemble more closely to reality, always respecting the relationship between rainfall intensity and rainfall volume. One of the most recent calculation methods is the G2P "Two-Parameter Gamma type Storm" (García-Bartual & Andrés-Doménech, 2017).

The G2P method was performed by analysing data from the city of Valencia and has been shown to be extendable to the Mediterranean area of which the study area is comprised. The authors propose a temporal design storm defined in analytical terms, using a gamma-type function with two parameters to be determined. The two parameters are estimated directly from 73 independent storms identified from rainfall records in the city of Valencia.

In particular, in order to assign a probability to the design storm, i.e. a return period, an auxiliary variable is introduced that combines the maximum intensity and the total accumulated rainfall. As a result, for a given return period, a set of three storms with different duration, depth and maximum intensity is defined. This process is explained in more detail hereafter.

The expression used with the method to determine rainfall intensities is by means of a continuous analytical function:

$$i(t) = i_0 * f(t)$$

where:

- $i(t)$ (mm/h) is the intensity of the storm at instant t
- i_0 (mm/h) is the instantaneous peak intensity of the storm
- t (min) is the time elapsed since the beginning of the storm
- f(t) is a gamma-type function defined by:

$$f(t) = \varphi * t * e^{1-\varphi t}$$

where φ (min^{-1}) is one of the parameters defining the method together with i(t).

The adopted function f(t) must reproduce the life cycle of the activity of a convective storm, i.e. an initial development until reaching the maturity phase, during which maximum intensities are reached, followed by a dissipation phase in time, typified by a progressive attenuation of rainfall.

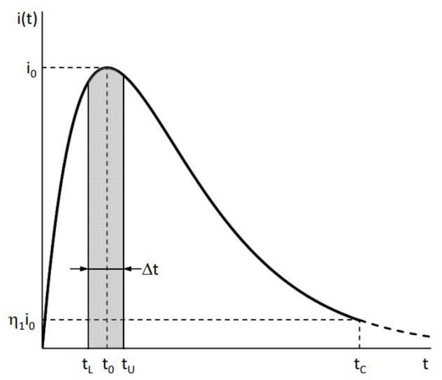

Figure 20. Typical design storm using the G2P method. Source: García-Bartual & Andrés-Doménech, 2017

The 73 real convective storms analysed in Valencia showed that there is an empirical correlation between the maximum intensity of the storm and the total accumulated rainfall:

$$\alpha_i = \frac{P}{I_{10}}$$

where:

- P (mm) is the total accumulated rainfall
- I_{10} (mm/h) is the maximum intensity of the storm with a ten-minute separation level

Following this correlation analysis, 3 sets of different events were identified, according to their duration. With this, it was concluded that for any given return period three different families are associated, all of them with equivalent magnitudes, but with different intensity, time and volume patterns.

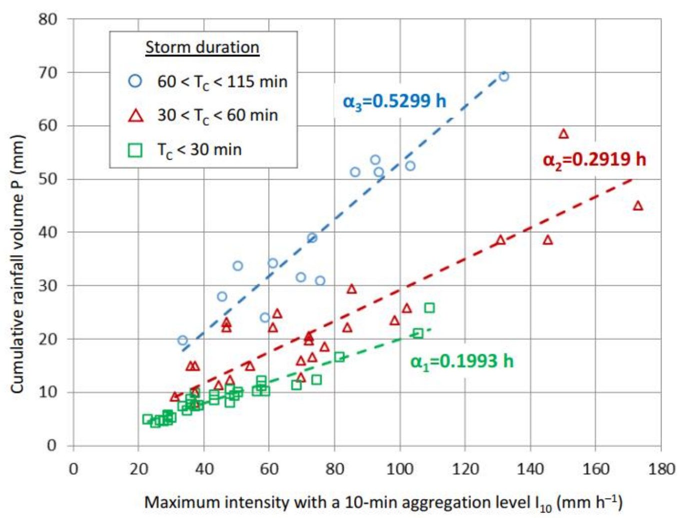

Figure 21. Correlation between maximum precipitation and maximum intensity as a function of the event duration. Source: García-Bartual & Andrés-Doménech, 2017

The design storm families for the study that have been considered correspond to 20, 40 and 70 minutes for the short-, medium- and long-term storms, respectively. As indicated above, the return period that should be associated with the design downpours is 15 years.

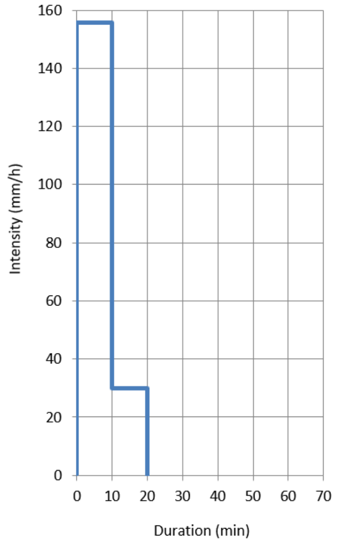

Figure 22. Hyetogram of short-term design storm. T=15 years

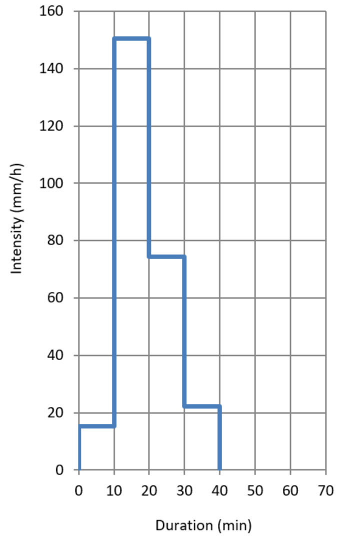

Figure 23. Hyetogram of medium-term design storm. T=15 years

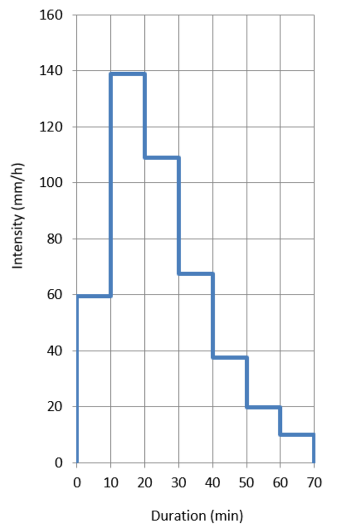

Figure 24. Hyetogram of long-term design storm. T=15 years

The characteristics of each of the design storms hyetograms are given in the tables below:

Table 13. Hyetogram volume and intensity values for design storms T = 15 years

Block	Short-term design storm		Medium-term design storm		Long-term design storm	
	Intensity (mm/h)	Volume (mm)	Intensity (mm/h)	Volumen (mm)	Intensity (mm/h)	Volume (mm)
1	155,68	25,95	15,19	2,53	59,47	9,91
2	29,90	4,98	150,52	25,09	138,73	23,12
3			74,31	12,39	109,08	18,18
4			22,15	3,69	67,38	11,23
5					37,45	6,24
6					19,61	3,27
7					9,89	1,65

The maximum values of duration, intensity and volume related to each storm are presented in the table below.

Table 14. Maximum values on design storm's hyetograms

Design storm	Duration (min)	Intensity (mm/h)	Volume (mm)
Short-term	20	155,68	30,93
Medium-term	40	150,52	43,70
Long-term	70	138,73	73,60

5.3.3 Comparison of different design storms

After having determined the two types of design downpours to be used in this work, based on the IDF curve and those determined by the G2P method, a graph comparing the four design storms is presented below.

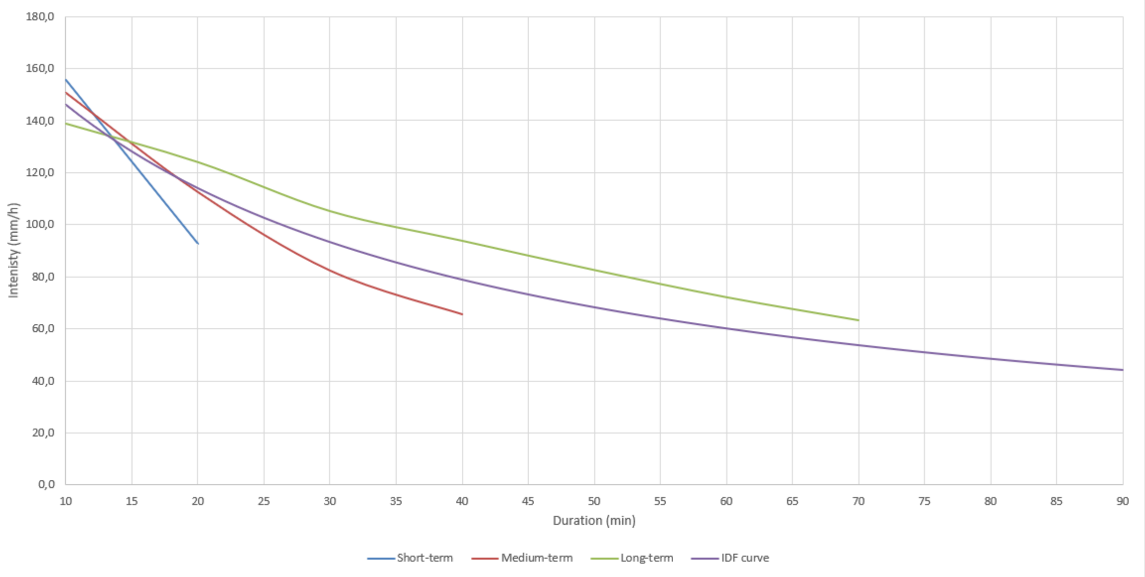

Figure 25. Comparison on design stroms T = 15 years

This shows that for long duration events, the IDF curves are on the unsafe side, identifying intensities much lower than those that could take place. Also, in general terms, the design shower that is closest to the IDF curve is the one of medium duration, a statement that can be compared in the modelling of the solutions later.

6 NATURAL STATE

As indicated in the problematic of the study, the aim of the Sustainable Drainage Systems in this project is, among others, to return the catchments of the study site to the natural state before the urbanisation of the area and impermeabilization.

To do this, the parameters related to the discharge point, the draining catchments and the rainfall-runoff transformation model used for the simulation of the site in its natural state must be known beforehand.

6.1 Discharge point

In the section on location and current situation, information has been provided on the behaviour of the Poyo basin, in which the study area is located. In this section, reference is made to La Saleta ravin, which is the closest hydraulic element to the study area and which, due to conditioning factors such as topography, is the optimum point for the discharge.

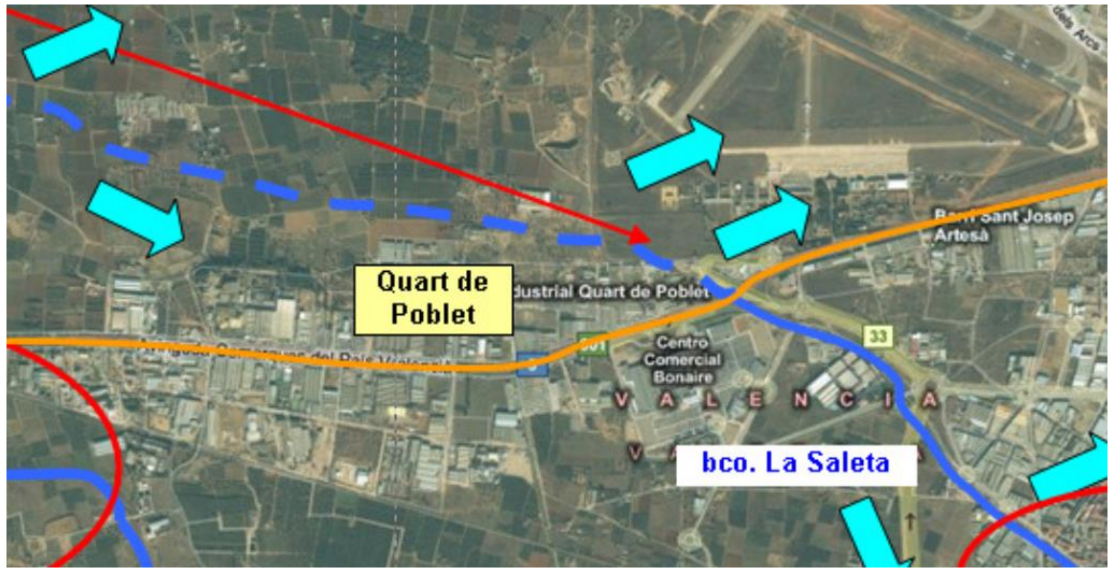

Figure 26. Location of La Saleta ravine and discharge point. Source: F.Franch et al., 2010

6.2 Catchments

The study area has been divided into three drainage catchment areas N1, N2 and N3.

Table 15. Catchments' surface

N1	N2	N3	TOTAL
156.712 m2	341.245 m2	696.606 m2	**1.194.563 m2**

At present, rainwater runoff from the catchment areas diverges naturally into La Saleta ravine without a drainage system, as shown in the following figure:

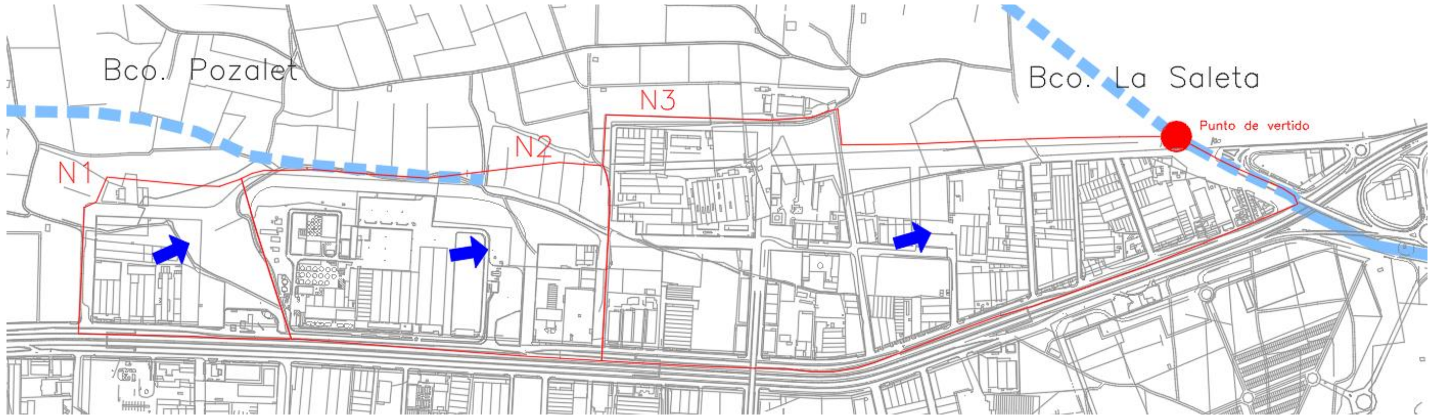

Figure 27. Catchments and discharge point

In the practical application of the study, the same discharge point for the runoff will be chosen as in the natural case since the runoff is discharged directly into the natural environment.

6.3 Rainfall-runoff transformation model

The estimate of peak natural flood flows is calculated from rainfall-runoff transformation models. Ideally, this information on peak flows should be provided by the CHJ, or any other competent Water Authority. In the absence of this data, it is predisposed to calculate the annual peak flow corresponding to a return period of 15 years.

The regulation Norma 5.2-IC Drenaje superficial, Orden FOM/298/2016 (Ministerio de Fomento, 2016) explains the methodology to be followed for the calculation of peak flows. The characteristics of the catchments, notably their size, and the return period considered determine that the rainfall-runoff transformation model to be used is the rational method. (A< 50 Km2 and Tr < 25 years).

The rational traditional method overestimates the flow frequency law, especially for low and medium return periods. Because of this, a methodology that preserves the advantages of the rational methodology, correcting the deficiencies of the method, was published in an article in the Civil Engineering journal for the Dirección General de Carreteras de España (Témez, 1992). This methodology is called the **modified Témez method.**

The modifications are made on the maximum daily rainfall and introduces the uniformity coefficient, leaving aside the majorization of 20%. In this way, it corrects the dependence of the runoff coefficient on the return period. The rest of the procedure is carried out according to regulation Norma 5.2-IC Drenaje superficial (Ministerio de Fomento, 2016).

The natural peak flow is calculated by the equation:

$$Q_T = \frac{I(T, t_c) * C * A * K_t}{3{,}6}$$

where:

- Q_T (m3/s) is the annual peak flow corresponding to the return period T.
- I(T, t_c) (mm/h) is the rainfall intensity corresponding to the considered return period T, for a storm duration equal to the time of concentration tc of the catchment.
- C (adimensional) is the average runoff coefficient of the catchment.
- A (km2) is the area of the catchment.
- K_t (adimensional) is the coefficient of uniformity in the temporal distribution of rainfall

6.3.1 Rainfall intensity

$$I(T, t_c) = I_d, F_{int}$$

where:

- I_d (mm/h) is the mean daily corrected rainfall intensity corresponding to the return period T

$$I_d = \frac{P_d * K_A}{24}$$

where P_d (mm) is the daily rainfall corresponding to a return period T, obtained from the most appropriate extreme function for the data of the area and K_A (adimensional) is the rainfall reduction factor per catchment area. The reduction factor is equal to 1 when the catchment area is less than 1 km2, as is the case for all draining catchments. Otherwise:

$$K_A = 1 - \frac{log_{10}(A)}{15}$$

- F_{int} (adimensional) is the intensity factor that introduces the torrential rainfall to the study area and is calculated as the maximum of two values:

$$F_{int} = \max{(F_a; F_b)}$$

where F_a is the factor obtained from the torrentiality index I_1/I_d that expresses the ratio between the hourly rainfall intensity and the corrected daily mean.

$$F_a = (\frac{I_1}{I_d})^{3,5287-2,5287*t^{0,1}}$$

The value of the torrentiality index varies according to the geographical area. In the east of the peninsula, coinciding with the Mediterranean area, the torrential index is 11.

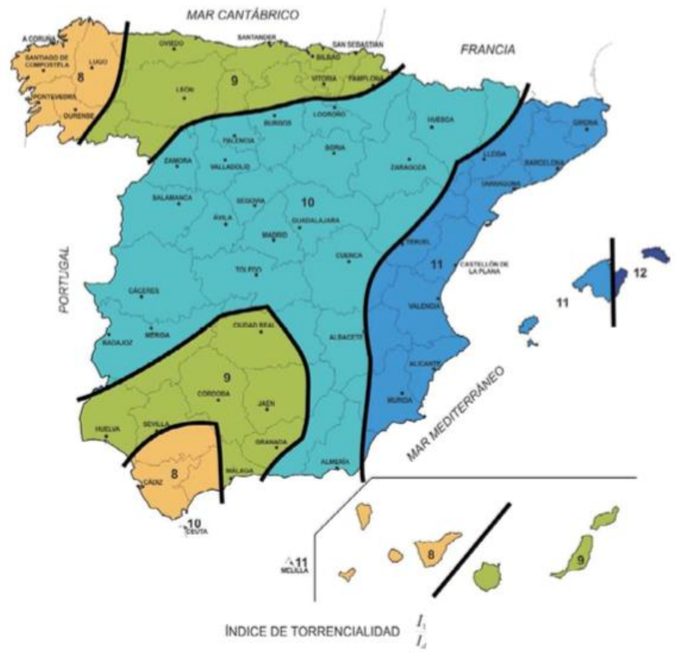

Figure 28. Torrential indez map. Source: Ministerio de fomento, 2016

Where Fb is the factor obtained from the IDF curves that relates the rainfall intensity corresponding to a return period T and a time of concentration t_c (IDF(T,t_c)) and the rainfall intensity corresponding to a return period T and a storm time equal to 24h (IDF(T,24)). This is multiplied by a factor K_b which considers the ratio between the maximum annual daily intensity.

$$F_b = K_b * \frac{I_{IDF}(T, t_c)}{I_{IDF}(T, 24)}$$

From the IDF curve we obtain that the intensity in a rainfall time of 24 h is 6.4 mm/h. For K_b, in the absence of a specific calculation, 1.13 is taken.

6.3.2 Concentration time

The concentration time t_c is the minimum time required for the entire surface of the catchment to be contributing runoff, from the beginning of the rainfall. It is obtained by calculating the longest travel time from any point in the catchment to the point of discharge.

The empirical formula of Témez, obtained from a modification of that of the US Army Corps of Engineers indicates that the time of concentration tc (h) is:

$$t_c = 0,3 * (\frac{L_c}{J_c^{0,25}})^{0,76}$$

Where:
- L_c (km) is the length of the watercourse
- J_c (adimensional) is the average slope of the watercourse

6.3.3 Runoff coefficient C

The runoff coefficient is calculated from runoff production models. Temez's method is based on the production model of the US SCS (Soil Conservation Service) modifying them for the Spanish territory.

$$\text{If } P_d * K_A > P_0 \qquad C = \frac{\left(\frac{P_d*K_A}{P_0}-1\right)*\left(\frac{P_d*K_A}{P_0}+23\right)}{\left(\frac{P_d*K_A}{P_0}+11\right)^2}$$

$$\text{If } P_d * K_A \leq P_0 \qquad C = 0$$

where:
- C (adimensional) is the runoff coefficient.
- P_d (mm) is the daily rainfall corresponding to the return period T
- K_a (adimensional) is the rainfall reduction factor per area, calculated as 1 due to the small size of the catchment.
- P_0 (mm) is the runoff threshold.

The runoff threshold represents the amount in mm of minimum rainfall that must occur for runoff generation to begin.

$$P_0 = P_0^i * \beta$$

where P_0^i is the initial value of the runoff threshold and β a runoff threshold correction coefficient.

The initial value of the runoff threshold is tabulated in Regulation 5.2-IC depending on land cover, use, slope and land type. The land of Quart de Poblet belongs to hydrological group C, as can be seen in the map of hydrological land groups. This hydrological group has the characteristics of very slow infiltration when very wet and imperfect drainage.

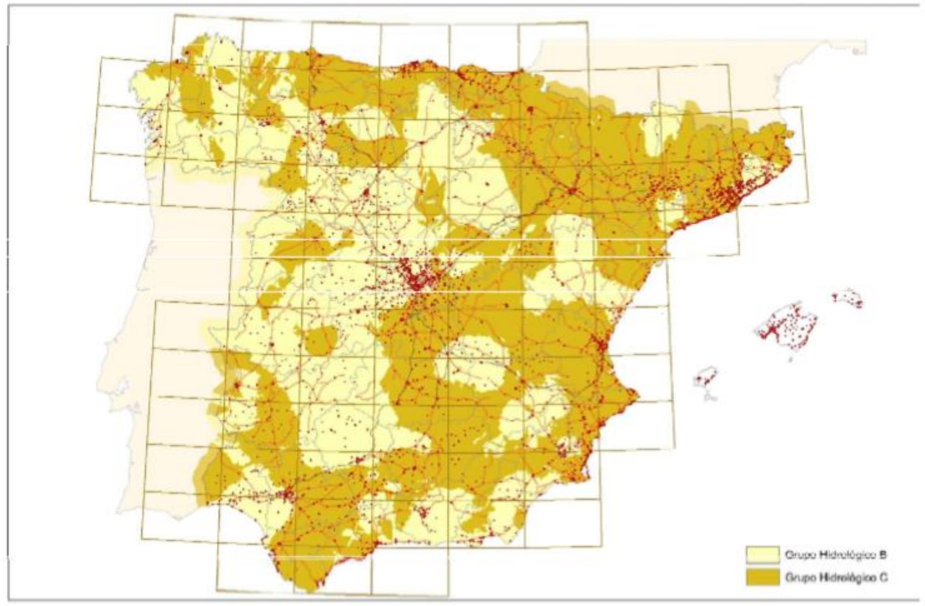

Figure 29. Hydrological land groups map. Source: Ministerio de fomento, 2016

In determining the value in its natural state, it has been considered according to the classification provided by the European Corine Land Cover project. The cartographic viewer of the Generalitat Valenciana allows the use of soils to be superimposed on the cartographic maps. This tool has been accessed to find out the current land use.

Currently, the land is mainly used for "Industrial or commercial areas" (12100) in purple, with small parts of "Natural pastures" (32100) in green.

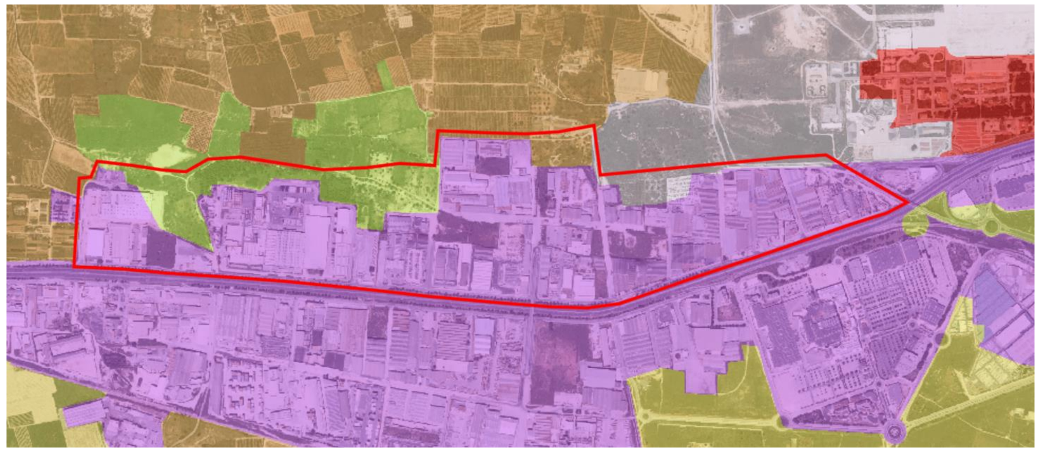

Figure 30. Land uses by Corine Land Cover 2018. Source: GVE map viewer

For the study of the runoff threshold in a natural state, the soil before urbanisation and industrialisation of the area has been considered. Therefore, the soil was determined as "Mosaic of irrigated agricultural crops with significant areas of natural and semi-natural vegetation" (24320) with a hydrological group between B/C and a slope of less than 3%, which corresponds to an initial value of the runoff threshold of between 25 mm and 16 mm. An intermediate value of $\boldsymbol{P_0^i}$ **= 20 mm** has been chosen.

The runoff threshold correction coefficient β can be obtained from the method given in Regulation 5.2-IC. However, in the case of obtaining enough representative flow data, a calibration must be made by comparison between real data and the results of the standard. For this reason, after intensive experimental studies, a new map has been determined that assigns generally more restrictive values throughout the Spanish peninsular territory (Lamberti et al., 2002). For the Valencia area, the coefficient is equal to 1.60, a value that is more restrictive than Regulation 5.2-IC.

6.3.4 Uniform temporal coefficient K

The coefficient K_t (adimensional) considers the correction due to the consideration of temporal uniformity of rainfall as the catchment size increases from the catchment concentration time t_c (h).

$$K_t = 1 + \frac{{t_c}^{1,25}}{{t_c}^{1,25} + 14}$$

6.3.5 Peak flow in natural state

By applying the modified Témez method, peak flows are determined for each catchment individually and for the entire study area. It would be incorrect to interpret the peak flow of the study area as the addition of the three catchments because, due to the different characteristics of the catchments, the peak flows of each catchment do not occur at the same time and adding them together would be too conservative.

Table 16. Peak flows in natural state

	N1	N2	N3	TOTAL
Q (m3/s)	1,77	3,25	4,40	**6,88**

In the following table, all the intermediate values that have allowed the calculation of the peak flow are shown:

Table 17. Parameters for rainfall-runoff transformation model

	N1	N2	N3	TOTAL
Pd (mm)	135,02	135,02	135,02	134,33
Ka	1	1	1	0,99
Id (mm/h)	5,63	5,63	5,63	5,60
Jc	0,00923	0,01733	0,00442	0,00970
Lc (Km)	0,46	0,81	1,58	2,37
tc (h)	0,40	0,55	1,19	1,40
Fa	18,66	15,62	9,88	8,96
Fb	18,53	15,51	9,81	8,79
Fint	18,66	15,62	9,88	8,96
i1/id	11	11	11	11
it/id	18,66	15,62	9,88	8,96
P0,i (mm)	20	20	20	20
Beta	1,6	2,6	3,6	5,6
P0 (mm)	32	32	32	32
A (km2)	0,16	0,34	0,70	1,19
I (mm/h)	105,0	87,9	55,6	50,1
C	0,378	0,378	0,378	0,377
Kt	1,022	1,033	1,082	1,098
Q (m3/s)	**1,77**	**3,25**	**4,40**	**6,88**

ANNEX Nº3: SOLUTIONS PROPOSAL

SUSTAINABLE DRAINAGE SYSTEMS (SUDS) IN INDUSTRIAL AREAS:
APPLICATION TO THE INDUSTRIAL AREA IN QUART DE POBLET (VALENCIA)

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Summary

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1 OBJECT

This annex aims to present the different solutions adopted after the design process. To this end, firstly, all the design criteria based on different regulations and technical documents are presented, which will be used to design each of the solutions and carry out the hydrological and hydraulic calculations that will support them.

Subsequently, the main alternatives that have been designed to meet the objectives of the work will be presented and, once these alternatives have been determined, they will be modelled using the SWMM software tool. The modelling will allow checking the correct functioning of the adopted solutions, as well as the optimisation of the design according to the case. In addition to being able to check the degree of satisfaction with respect to the proposed objectives.

2 DESIGN CRITERIA

2.1 Collector system network

The municipality of Quart de Poblet has no legislation regulating the criteria for the design, calculation and construction of collector systems. For this reason, it has been decided to use the legislation in application in the city of Valencia, which is nearby and has very similar rainfall events. The regulations used are the Regulations for urban drainage and sewerage works in the city of Valencia (Ayuntamiento de Valencia, 2015).

2.1.1 Hydraulic design

To obtain the hydraulic design, the design flow must be known, the collector must be dimensioned for the given design flow and various checks must be carried out in relation to velocities, energy line, arrangements and general hydraulic behaviour.

The decisive design criteria used for the hydraulic design are:

- The flow system shall preferably be **by gravity**, avoiding as far as possible continuous lifting and pumping systems.
- The operation of the collector shall be in a **uniform stationary regime.**
- The collector regime will be slow regime, limiting the Froude number to $\boldsymbol{Fr < 1}$.. As, in any case, the stable slow regime is assured when $Fr < 0{,}8$, we will try to limit the Froude number to this value.
- The collector will operate at **80% of the full section depth**, as this is the optimum operating capacity.
- The maximum velocity shall be limited to $V\boldsymbol{max = 4\ m/s}$ to avoid friction damage.
- The minimum velocity shall be limited to a $\boldsymbol{Vmin = 1{,}2\ m/s}$ to avoid sedimentation of entrained suspended solids and clogging.
- The minimum internal diameter to avoid clogging and to facilitate cleaning is $\boldsymbol{Dmin = 335\ mm}$.
- The frictional energy loss equation is given by the **Manning's equation**.
- The Manning's number of PVC and concrete is $\boldsymbol{n = 0{,}011}$, a conservative value to consider the degradation suffered by the collector.
- **Continuity** will be ensured for **the top line and the energy line**. Therefore, the collector flushing will be done obligatorily through the top line.
- The **energy line** will always be located **below the ground level** and backwaters with a lower energy downstream than the energy upstream will be avoided.
- In the case of the existence of hydraulic backwater, it shall be downstream at a distance of $\boldsymbol{d < 10\emptyset}$

2.2 Sustainable drainage systems

The design criteria for the SuDS network are based on international references, specifically the SuDS Manual provided by the United Kingdom [40]. As indicated above, there are no Spanish regulations governing these drainage systems. The recommendations of this manual are therefore taken as a reference. In the technical context of SuDS, the different functions of these and the advantages they provide when implemented have been explained in detail. In this section, it will be mainly mentioned the design criteria that are followed to achieve the quantity and quality control objectives.

The design criteria for the network of sustainable drainage systems are based on volumetric control, to achieve the objectives of quantity and quality control. The volumetric control of runoff is necessary to achieve the lamination, but not only for this reason, the control of the volume of runoff allows at the same time a quality control of the runoff water that will be discharged.

2.2.1 Quantity control

The quantity criteria to be treated to achieve the lamination of the flow generated by rainfall events is given by the V_{80} value. The value of V_{80} indicates that in 80% of the events a rainfall value lower than that indicated will be estimated. This value is obtained from the IDF curve calculated in the pluviometric analysis for the 15-year return period.

The choice as a quantity criterion is the V_{80} value, in The SuDS Manual (Woods Ballard et al., 2015) is defined as the volume of runoff that must be fully treated. These events are those that occur with greater occurrence and lesser entity, leaving all those events that are exceeded to be partially treated and avoiding causing unacceptable damage downstream or in the system itself.

The V_{80} value calculated in the pluviometric analysis is **15 mm.**

In addition, the aim is to maintain the natural drainage pattern of the area, trying to ensure that the evacuation flows to the discharge point do not exceed the flows calculated in a natural state for a return period of 15 years. This objective is much more ambitious and will therefore have larger storage volumes to manage.

An analysis is made with the three design storms and it is determined, as expected, that the one that will generate the largest volume will be the long downpour with **73.6 mm.**

Table 1. V80 value and maximum storage volume for T = 15 years

	N1	N2	N3
Storage volume V80 (m3)	1.988	4.609	9.927
Storage volume of long-term design storm (m3)	9.756	22.617	48.707

2.2.2 Quality control

The quality criterion indicated in the manual is to be able to store a runoff volume corresponding to the V_{90} value. The V_{90} value indicates that in 90% of the events a rainfall will be estimated to be less than the indicated value.

As already demonstrated in the rainfall analysis, the V_{90} value is **25 mm.**

The choice of the V_{90} value as a quality criterion is justified because it is defined as the volume of runoff that must be treated so that the water returning to the receiving medium or infiltrating into the ground is sufficiently clean so that it does not cause pollution of the medium.

Table 2. V90 storage volume T = 15 years

	N1	N2	N3
Storage volume V90 (m3)	3314	7682	16544

This method is based on associating the concentration of pollution with a volume of stored runoff. One of the disadvantages of this method is that it does not differentiate between the different qualities of runoff water that can be generated and would treat urban and industrial runoff in the same way, which have been shown not to have the same concentration of pollutants. Furthermore, it does not consider the different types of systems and their ability to mitigate pollutants.

As we work in an industrial environment where pollution is high, it is preferable to complement the V90 method with other methods that analyze the concentration of pollutants in more detail and thus ensure that good quality control is being exercised. The method that does this is the index mitigation method explained in the technical context and which will be developed in the proposed solutions.

Therefore, a design will be carried out to try to meet both the V90 quality objective, related to the volume of rainfall, and the pollutant mitigation quality objective, related to the layout of the sustainable drainage systems and the characteristics of the study area.

3 DESIGN PARAMETERS

3.1 Permeability coefficients

One of the most important factors when recognising the terrain under study is the permeability of the soil, because the determination of whether or not to use infiltration as a drainage technique for the proposed sustainable drainage system will depend substantially on it.

As indicated in *Annex Nº2: Initial data and design conditions*, the permeability of the study area is high and has been considered to be 12.9 mm/h after applying the safety coefficient of 1.5 proposed by the SUDS manual (Woods Ballard et al., 2015).

$$Kc = \mathbf{12,9\ mm/h}$$

3.2 Pluviometric data

Following on from *Annex Nº2: Initial data and design constraints*, a detailed explanation has been given of the rainfall analysis carried out to obtain all the values necessary for the development of this work. This section summarizes the most necessary rainfall data for the design of the drainage system.

The daily rainfall percentiles of the data series analyzed are as follows:

- On 80% of rainy days, rainfall does not exceed 15 mm $V_{80} = \mathbf{15mm}$
- On 90% of rainy days, rainfall does not exceed 25 mm $V_{90} = \mathbf{25mm}$

The maximum annual daily rainfall is obtained by the distribution that most closely resembles the real data, i.e. the SQRT-ET MAX distribution. The return period chosen is 15 years as it appears in the PATRICOVA indications. From this we obtain:

$$Pd_{max}(Tr = 15\ years) = \mathbf{135,02\ mm}$$

The design rainfall obtained by the G2P method results in three design storms of different durations. The values of maximum intensity and rainfall volume for the three design showers are represented in the following table:

Table 3. Design storms parameters

	Duration (min)	Intensity (mm/h)	Volume (mm)
Short-term	20	155,68	30,93
Medium-term	40	150,52	43,70
Long-term	70	138,73	73,60

3.3 Collectors network

The collectors network will be made of two different materials depending on the size. Pipes with a diameter of 1200 mm or less will be made of corrugated PVC material. The diameters chosen have been obtained from SANECOR (SANECOR) manufacturer's catalogue.

In pipes where diameters larger than 1200 mm are required, RIBLOC (RIBLOC) has been used as the reference manufacturer. The CONCRETLOC series is made up of rigid PVC pipes, helically shaped with a profile that has a wall structured by means of "T" shaped stiffeners and reinforcement with galvanised steel profile, giving the pipe a high circumferential rigidity. The diameters used for the project will be:

Table 4. Pipe network diameters

DN (mm)	Ø External (mm)	Ø Internal (mm)
400	400	364
500	500	452
630	630	590
800	800	775
1000	1000	970
1200	1200	1103
1300	1300	1268
1400	1400	1368
1500	1500	1468
1600	1600	1568
1700	1700	1668
1800	1800	1768
1900	1900	1868
2000	2000	1968
2100	2100	2068
2200	2200	2168
2300	2300	2268
2400	2400	2368
2500	2500	2468
2600	2600	2568
2700	2700	2668
2800	2800	2768
2900	2900	2868
3000	3000	2968

3.3.1 Complementary elements

The main complementary element to the collector network will be the manholes. The manholes will be placed at the beginning of each section, at intersections between two or more sections, when there is a change of direction or slope and along each section, ensuring a separation of no more than 100 metres, to facilitate maintenance, inspection and cleaning tasks.

In accordance with the Regulations for sewerage and urban drainage works in the city of Valencia (Ayuntamiento de Valencia, 2015), manholes with diameters of between 400 and 1500 mm will be installed for collectors with diameters of between 1000 and 1500 mm. In the case of collectors larger than 1500 mm in diameter, registration wells will be installed to connect the different collectors.

4 SOLUTIONS PROPOSAL

This section presents the four alternatives that have been designed to solve the problem of the study and that will serve as input for the subsequent verification by modelling.

The proposed alternatives are presented as follows:

- The first alternative consists of implementing the entire drainage system in a conventional system with a collectors’ network.
- The second alternative seeks to improve the response of the system in the previous case by proposing the design of a storm tank to retain and laminate the water during periods of rainfall and then introduce it into the sewerage network in a controlled way.
- The third alternative, based on the collector system of the first alternative, adds sustainable drainage systems such as pervious pavements, rain gardens and infiltration basins to improve the hydrological and hydraulic response of the catchments.
- The fourth and final alternative, to be designed in the case that the third alternative fails to meet the proposed threshold discharge flow, will improve the system response of the third alternative by adding a storm tank.

Before developing each of the proposals, the aspects and parameters that are common to all of them will be presented.

4.1 Commons aspects of the solutions

4.1.1 Subcatchment division

Common to all the scenarios is the division of the study area into sub-catchments. Based on the PGOU, it has been possible to divide the site plan of the area studied into three types of subcatchments, the street subcatchments with the C label, the private parcel with the PR label and the green zone parcel with the VE label. In this way each one has been identified with a different runoff coefficient.

It should be noted that for alternatives 3 and 4 where sustainable drainage systems are implemented, these will be positioned on the road or the green area, so the subcatchments will vary slightly in the case of the road, as well as its runoff coefficient. These differences will be explained in more detail in the explanation of the alternatives.

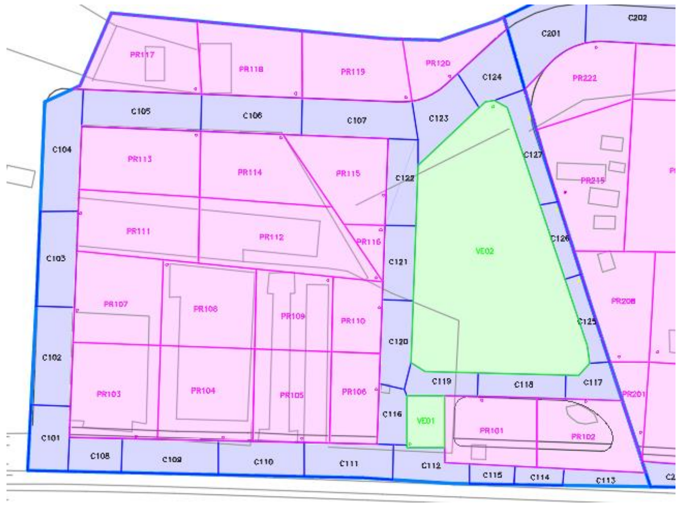


Figure 1. Subcatchment division on catchment N1

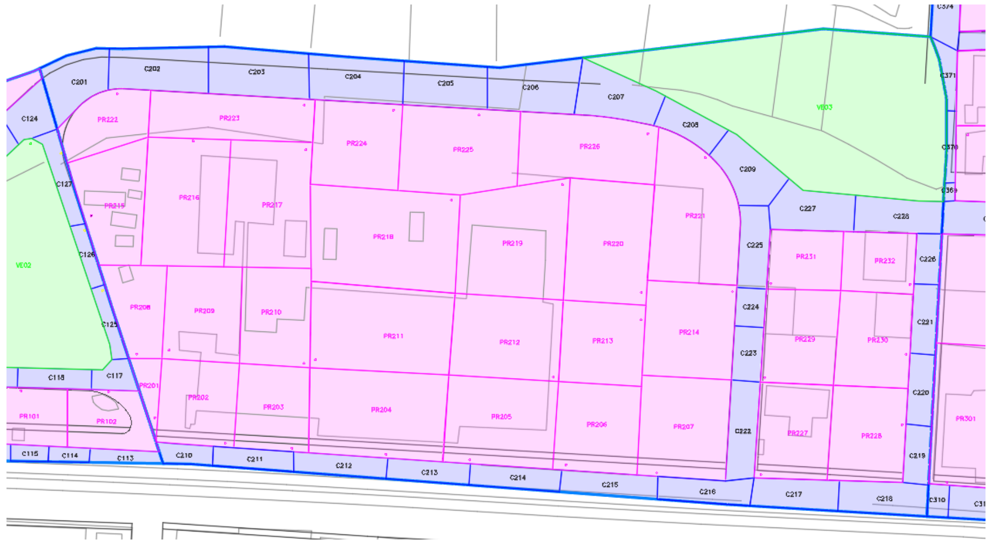


Figure 2. Subcatchment division on catchment N2

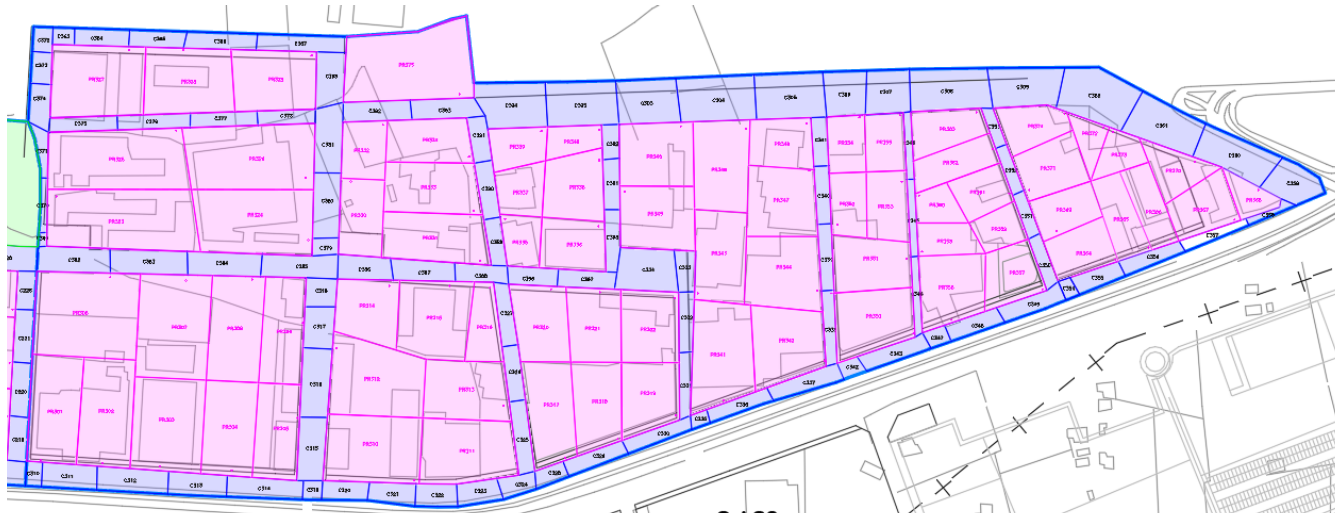


Figure 3. Subcatchment division on catchment N3

4.1.2 Runoff coefficient after urbanisation

One of the biggest hydrological problems with the urbanisation of areas is the increase in the impermeability of the soil, creating a modification in the hydrological behaviour. This causes runoff to be much greater, which can create material or environmental damage due to flooding.

Soil impermeabilization leads to changes in the runoff coefficient. The more permeable the soil is, the lower the coefficient will be and therefore the less runoff will be generated.

The valencian regulations (Ayuntamiento de Valencia, 2015) define the following runoff coefficients according to the basic surface types:

Table 5. Runoff coefficient for different surface

Surface basic type	Runoff coefficient
Impermeable	0,95
Edification	0,75
Permeable	0,05-0,30
Not-connected	0,00

Table 6. Runoff coefficient for types of surface

Surface type	Runoff coefficient
Big paved areas	0,95
Urban areas	0,85
Residential areas	0,5
Non.paved areas	0,05-0,3

Where:

- Big paved areas are large parking areas and large squares without gardens.
- Urban areas are areas consisting of streets, small squares and high-rise buildings.
- Residential areas are residential developments where single-family buildings are mixed with gardens.
- Non-paved areas are parks and gardens. This coefficient depends on the slope area:

Table 7. Runoff coefficient on not-paved areas

<table>
<tr><th rowspan="2">S < 2000 m2</th><th rowspan="2">2000 m2 < S < 5000 m2</th><th colspan="2">S > 5000 m2</th></tr>
<tr><td>Above road level</td><td>Below road level</td></tr>
<tr><td rowspan="2">0,3</td><td rowspan="2">0,2</td><td>0,2</td><td>0,05-0,1</td></tr>
</table>

In any case, the classification of the four surface classes will be made taking into account the General Urban Development Plan (PGOU) in application in the industrial area. Without taking into account the current situation, which varies slightly.

The runoff coefficients corresponding to the sustainable drainage systems have also been identified.

Table 8. Runoff coefficient on SuDS

Surface	Runoff coefficient
Pervious pavement	0,70
Rain garden	0,30
Infiltration basin	0,30

The pre-dimensioning of the solutions will be carried out following these determined runoff coefficients, for a first approximation. However, a verification and validation will be carried out with the SWMM software to be able to reproduce the rainfall-runoff process using another method, with the curve number parameter, and thus carry out a correct design of the network. The modelling with the SWMM software is detailed below.

The average runoff coefficient for each catchment, in each alternative, has been determined for information purposes to get an idea of the general imperviousness of the study area. However, this average coefficient is not actually used to determine the hydrological response, since the data entered in the flow calculation formula is that corresponding to the runoff coefficient of each sub-basin linked to the collector section.

Table 9. Average runoff coefficients for each catchment

Catchment	Alternatives 1 y 2	Alternatives 3 y 4
N1	0,703	0,693
N2	0,716	0,712
N3	0,804	0,777
TOTAL	**0,766**	**0,748**

4.2 Alternative 1. Conventional network solution

The first alternative consists of implementing the entire drainage system in a conventional way with a network of collectors. As can be expected and will be explained in the results, this scenario cannot solve the main problem of the work related to the excessive discharge of water into La Saleta ravine. However, this scenario serves as a basis for comparison for the rest of the scenarios.

It cannot be considered as the alternative 0, which corresponds to no action, because this alternative is a substantial improvement on the current situation, as there is currently no drainage system in the study area and this alternative would improve the control and conduction of the runoff generated.

4.2.1 Plan layout

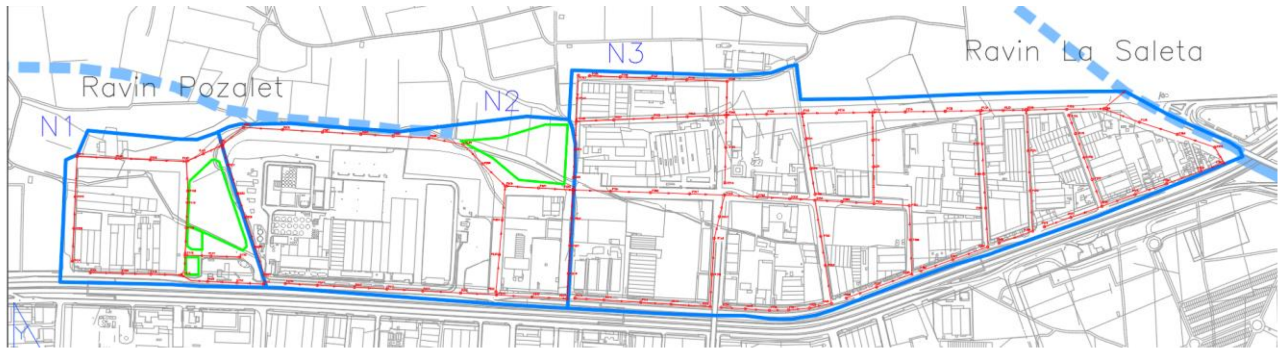


Figure 4. Alternative 1 layout

The plan layout of the conventional network has been carried out on the public subcatchments, as this study does not intend to design the drainage system of the private subcatchments. However, the study does consider the runoff from the private subcatchments in the estimations that will be added to the collector network.

The requirements for grouping the segments of the water line of a collector is determined by the Valencian townhall (Ayuntamiento de Valencia, 2002), in which it is indicated that the consecutive segments of the line must:

- Be made of the same material
- Have the same shape or type of section
- Conduct the same type of water
- Be in the same street
- Not have lateral junctions along their entire length

All the sections of line must always respect that there are no bifurcations downstream but that, in any case, two sections of collectors converge in a collector downstream.

The solution with a conventional drainage system means that all the flow discharged into the receptor environment is discharged into La Saleta ravine, at the most north-westerly point of the study area. Therefore, whenever possible, the collectors will follow the direction from south to north and from west to east.
Taking all these parameters into account, a layout of the collectors is made for the three catchment areas. A distinction is made between catchment areas N1, N2 and N3 to be able to make a global analysis of the whole and a local analysis in each of the catchment areas.

The following figures show schematically the segments chosen for the different catchment areas, where catchment area N1 has 7 segments of collectors, catchment area N2 has 5 segments of collectors and, finally, catchment area N3, which is the largest, has 24 segments of collectors.

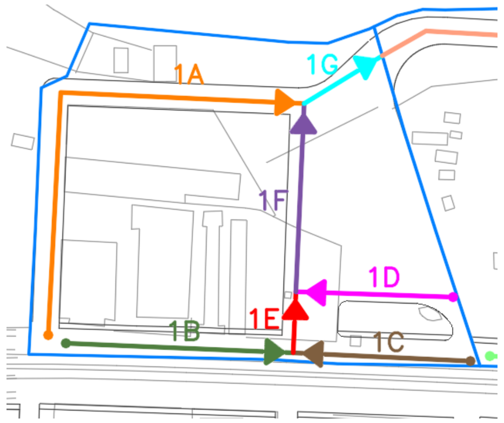


Figure 5. Collectors' segments on catchment N1

Figure 6. Collectors' segments on catchment N2

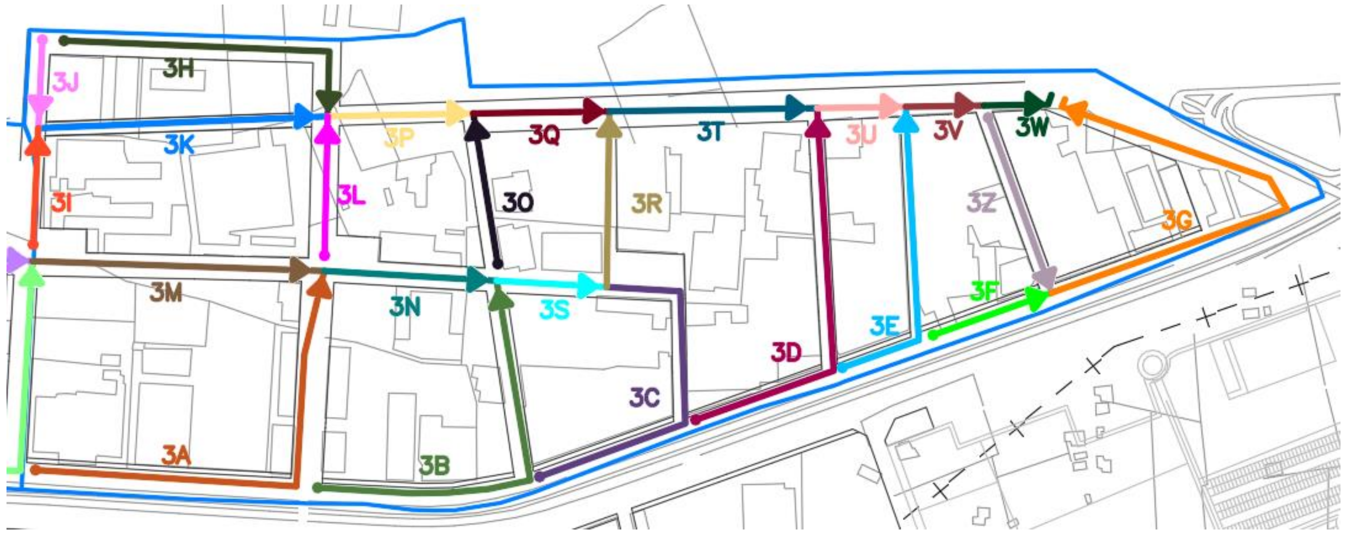


Figure 7. Collectors' segments on catchment N3

The directions followed by the network of collectors have been chosen mainly to reduce the water flow through them as much as possible, as well as trying to follow the natural slope of the land to respect the minimum overflow limits without having to use large slopes in the collectors.

4.2.2 Hydrological calculation

The method used for the calculation of design flows in each section of the drainage system is the Rational Calibrated Method, included in the regulations for drainage and urban drainage works in the city of Valencia (Ayuntamiento de Valencia, 2015). This method is a variant of the rational method adapted to the Valencia area.

This method indicates that the design flow of the collectors will be:

$$Q = \frac{K_p * I * \sum_{i=1}^{n} C_i * A_i}{360}$$

Where:

- Ai (ha) is the surface area.
- I (mm/h) is the design storm intensity corresponding to 15 years return period.
- Ci (adimensional) is the runoff coefficient of the surface area.
- Kp (adimensional) is the catchment spreading coefficient, value 1 is taken.

Indicating that, if the design flow obtained in the above expression represents a reduction of more than 5% with respect to the flow of the upstream connected section, the flow of the upstream section, reduced by 5%, will be adopted as the design flow. This avoids sizing in the event of an overlap of peak flows not considered in the Rational Method.

4.2.2.1 Design storm intensity

The design storm intensity is obtained from the return period IDF curve, previously determined in the input data, which has for each duration value an assigned intensity.

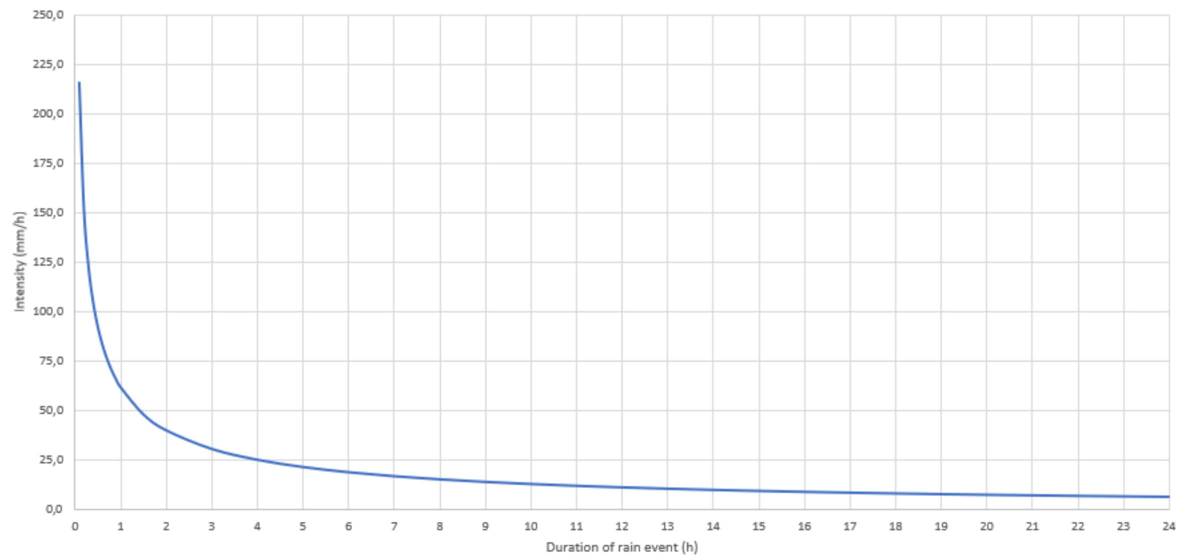


Figure 8. IDF curve for T=15 years

The intensity-related duration value for each segment is calculated as the time of concentration.

$$tc = ts + \frac{\alpha}{60} \sum_{i=1}^{n} \frac{L_i}{V_i}$$

Where:

- n (adimensional) is the number of upstream collectors' segments.
- Li (m) is the length of each collector segment.
- Vi (m/s) is the velocity in each collector segment, calculated with the uniform flow assumption and with the design flow rate of each segment.

- ts (min) is the surface travel time which takes the maximum value between 6 minutes and L0/V0 where L0 is the length (m) from the furthest point of the catchment to the first collector and V0 (m/s) is the surface velocity which can be approximated to be halfway along the first collector.
- α (adimensional) is the network travel time factor which considers that the collectors do not run at full flow all the time and has a value of 1.2 in Valencia.

If the time of concentration is less than 10 minutes, the duration value of 10 minutes and its corresponding intensity will be adopted directly.

4.2.3 Hydraulic calculation of transport infrastructures

Once the layout of the collectors has been decided, both in plan and longitudinal profile, we will calculate the necessary diameter to be able to manage the peak flow for a return period of 15 years.

The design of the collectors is based on the knowledge of the peak flow by the Rational Calibrated Method, as indicated in the hydrological calculation.

We assume that the flow within the network is uniform. The equation describing the flow in a uniform regime flow is:

$$Q = \frac{S_m}{n} * \left(\frac{S_m}{P_m}\right)^{\frac{2}{3}} * I^{\frac{1}{2}}$$

where:
- Q (m3/s) is the flow rate through the collector.
- S_m (m2) is the wetted section of the collector.
- P_m (m) is the wetted perimetre of the collector.
- n (adimensional) is the Manning's roughness coefficient.
- I (m/m) is the collector slope.

Knowing most of the values, we still need to know the diameter of the collectors, as well as their normal depth, which would influence the section and wetted perimetre. In the design criteria of the solution, it has been indicated that the collectors will work at 80% of their hydraulic capacity, this being the optimum draught.

Thus, substituting the values for the parameters of the circular section, we obtain:

$$Q = \frac{k}{n} * D^{\frac{8}{3}} * I^{\frac{1}{2}}$$

Where:
- D (m) is the diameter of the collector.
- k (adimensional) is a value for circular conductors taken from the flow-diameter relation

Y/D	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0.0	-	0.000047	0.00021	0.00050	0.00093	0.00150	0.00221	0.00306	0.00407	0.00521
0.1	0.00651	0.00795	0.00953	0.0113	0.0131	0.0152	0.0173	0.0196	0.0220	0.0246
0.2	0.0273	0.0301	0.0331	0.0362	0.0394	0.0427	0.0461	0.0497	0.0534	0.0572
0.3	0.0610	0.0650	0.0691	0.0733	0.0776	0.0820	0.0864	0.0910	0.0956	0.1003
0.4	0.1050	0.1099	0.1148	0.1197	0.1248	0.1298	0.1349	0.1401	0.1453	0.1506
0.5	0.156	0.161	0.166	0.172	0.177	0.183	0.188	0.193	0.199	0.204
0.6	0.209	0.215	0.220	0.225	0.231	0.236	0.241	0.246	0.251	0.256
0.7	0.261	0.266	0.271	0.275	0.280	0.284	0.289	0.293	0.297	0.301
0.8	0.305	0.0308	0.312	0.315	0.318	0.321	0.324	0.326	0.329	0.331
0.9	0.332	0.334	0.335	0.335	0.335	0.335	0.334	0.332	0.329	0.325
1.0	0.0312	-	-	-	-	-	-	-	-	-

Figure 9.K values for flow-diameter relation. Source: Universidad de Salamanca

For a ratio of 0.8 we obtain a coefficient k of 0.305. The diameter will be calculated then:

$$D = \left(\frac{Q * n}{I^{\frac{1}{2}} * 0{,}305}\right)^{\frac{3}{8}}$$

Once the design diameter is known, before the design can be validated, it is important to verify that the velocity limits, both upper and lower, as well as the power line limit, are respected.

The maximum velocity in rainwater collectors is limited to 4 m/s to avoid damage to the pipes and the minimum velocity is limited to 1.2 m/s to avoid sedimentation and to allow self-cleaning of the collector.

The velocity of a circular collector is calculated with Manning's energy loss equation assuming uniform flow:

$$V = \frac{8 * Q}{D^2 * (\theta - sen\theta)}$$

Where:
- V (m/s) is the flow velocity.
- D (m) is the diameter of the collector.
- Q (m3/s) is the flow rate.
- θ (rad) is the wetted surface angle which is calculated by an iterative method from the equation:

$$(\theta - sen\theta)^5 - \theta^2 * \frac{8192}{D^8} * \left(\frac{Q * n}{\sqrt{i}}\right)^3 = 0$$

The flow energy line shall be below always ground level. The energy line is calculated as:

$$H = z + y + \frac{V^2}{2 * g}$$

Where:

- H (m) is the power line elevation.
- z (m) is the baseplate height.
- y (m) is the normal depth corresponding to the design flow rate.
- V (m/s) is the velocity corresponding to the design flow rate.

The check shall be made by comparing the energy levels at the beginning and at the end of each segment with the corresponding ground levels.

The final hydraulic check of the design shall be made by means of the Froude number, which shall ensure a slow rate below 1, and as far as possible below 0.8 to be on the safe side in the statement of a stable slow rate.

4.2.4 Collectors design

As can be deduced from the methodology proposed, the hydrological calculation depends on the results obtained by the hydraulic calculation and vice versa. This means that in order to obtain the dimensioning of the collector network it is necessary to carry out an iterative process.

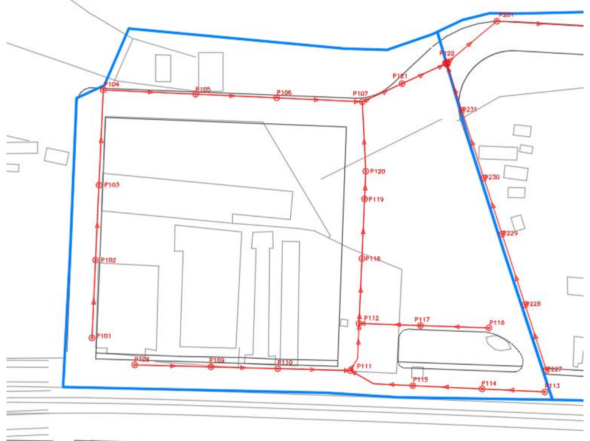


Figure 10.Collectors' design for catchment N1. Alternative 1

Figure 11. Collectors' design for catchment N2. Alternative 1

Figure 12. Collectors' design for catchment N3. Alternative 1

The results of the collector design for alternative 1 are presented in the following tables highlighting the most representative parameters. These design parameters will serve as the basis for the modelling of the scenario and subsequent verification.

Table 10. Design collectors results of catchment N1

N1 COLLECTORS' DESGIN												
Collector code			Collectors characteristics									
Segment	Upstream manhole	Downstream manhole	L	Q	Internal diameter	External diameter	Slope	V	Fr	Depth	Upstream covering	Downstream covering
-	-	-	m	m3/s	m	m	-	m/s	-	m	m	m
1A	**101**	**102**	80,38	0,163	0,452	0,500	0,0030	1,31	0,43	0,33	2,83	2,65
	102	**103**	77,49	0,409	0,775	0,800	0,0030	1,68	0,55	0,42	2,65	2,50
	103	**104**	98,36	0,831	0,970	1,000	0,0030	2,01	0,48	0,54	2,50	1,00
	104	**105**	95,31	0,943	0,970	1,000	0,0030	2,07	0,44	0,59	1,00	1,29
	105	**106**	83,52	1,319	0,970	1,000	0,0030	2,20	0,31	0,74	1,29	1,30
	106	**107**	88,45	1,667	1,103	1,200	0,0030	2,36	0,35	0,77	1,30	1,57
1B	**108**	**109**	78,68	0,294	0,590	0,630	0,0030	1,53	0,46	0,19	1,42	1,43
	109	**110**	68,94	0,544	0,775	0,800	0,0030	1,80	0,46	0,29	1,43	1,00
	110	**111**	74,82	0,761	0,775	0,800	0,0030	1,90	0,30	0,62	1,00	1,15
1C	**113**	**114**	64,85	0,054	0,364	0,400	0,0047	1,20	0,98	0,20	1,00	1,65
	114	**115**	71,43	0,076	0,364	0,400	0,0047	1,31	0,83	0,20	1,65	2,17
	115	**111**	67,99	0,133	0,452	0,500	0,0047	1,50	0,78	0,20	2,17	1,72
1D	**116**	**117**	70,56	0,229	0,452	0,500	0,0060	1,85	0,62	0,33	1,25	1,57
	117	**112**	63,35	0,399	0,590	0,630	0,0060	2,15	0,67	0,38	1,57	1,00
1E	**111**	**112**	50,65	0,952	0,970	1,000	0,0030	2,07	0,44	0,39	1,50	1,34
1F	**112**	**118**	67,12	1,500	1,103	1,200	0,0030	2,31	0,38	0,71	2,50	2,30
	118	**119**	61,56	1,720	1,103	1,200	0,0030	2,37	0,33	0,79	2,30	1,43
	119	**120**	28,48	2,008	1,268	1,300	0,0030	2,50	0,39	0,77	1,43	1,89
	120	**107**	71,91	2,098	1,268	1,300	0,0030	2,52	0,38	0,79	1,89	2,46
1G	**107**	**121**	45,09	3,930	1,468	1,500	0,0030	2,89	0,28	1,10	2,90	1,24
	121	**122**	50,29	4,080	1,468	1,500	0,0030	2,90	0,26	1,14	1,24	1,75

Table 11. Design collectors results of catchment N2

N2 COLLECTORS' DESGIN												
Collector code			Collectors characteristics									
Segment	Upstream manhole	Downstream manhole	L	Q	Internal diameter	External diameter	Slope	V	Fr	Depth	Upstream covering	Downstream covering
-	-	-	m	m3/s	m	m	-	m/s	-	m	m	m
2A	**122**	**201**	67,75	4,681	1,768	1,800	0,0015	2,32	0,18	1,37	1,48	3,61
	201	**202**	96,22	4,957	1,868	1,900	0,0015	2,38	0,20	1,35	3,61	4,15
	202	**203**	97,45	5,267	1,868	1,900	0,0015	2,40	0,18	1,41	4,15	6,30
	203	**204**	93,71	5,733	1,968	2,000	0,0015	2,47	0,19	1,42	6,30	5,32
	204	**205**	78,94	5,944	1,968	2,000	0,0015	2,48	0,18	1,47	5,32	5,94
	205	**206**	87,57	6,587	1,768	1,800	0,0035	3,51	0,30	1,24	5,94	3,16
	206	**207**	87,77	6,906	1,768	1,800	0,0035	3,53	0,28	1,29	3,16	1,30
	207	**208**	67,03	7,117	1,768	1,800	0,0035	3,54	0,27	1,33	1,30	1,47
	208	**209**	77,7	7,410	1,768	1,800	0,0035	3,56	0,25	1,27	1,47	1,70
2B	**210**	**211**	78,34	0,278	0,590	0,630	0,0040	1,70	0,62	0,31	5,40	5,32
	211	**212**	90,77	0,711	0,775	0,800	0,0040	2,13	0,48	0,31	5,32	5,30
	212	**213**	79,12	1,440	0,970	1,000	0,0040	2,52	0,39	0,34	5,30	5,41
	213	**214**	87,78	1,780	1,103	1,200	0,0040	2,68	0,43	0,24	5,41	5,33
	214	**215**	92,66	2,317	1,268	1,300	0,0040	2,88	0,45	0,32	5,33	4,37
	215	**216**	87,53	2,626	1,268	1,300	0,0040	2,96	0,41	0,50	4,37	4,23
	216	**217**	87,41	2,671	1,268	1,300	0,0040	2,96	0,40	0,49	4,23	4,29
	217	**218**	84,99	2,723	1,268	1,300	0,0040	2,97	0,39	0,49	4,29	3,48
	218	**219**	81,71	2,739	1,268	1,300	0,0045	3,12	0,44	0,30	3,48	3,22
	219	**220**	87,84	2,948	1,268	1,300	0,0045	3,17	0,41	0,25	3,22	2,95
	220	**221**	66,07	3,162	1,268	1,300	0,0045	3,20	0,37	0,48	2,95	2,28
	221	**226**	69,37	3,274	1,268	1,300	0,0045	3,22	0,35	0,47	2,28	1,44
2C	**222**	**223**	91,21	0,354	0,590	0,630	0,0030	1,58	0,36	0,35	3,31	3,82
	223	**224**	84,83	0,779	0,970	1,000	0,0030	1,98	0,50	0,30	3,82	1,50
	224	**209**	83,23	1,515	1,103	1,200	0,0030	2,32	0,38	0,35	1,50	1,85
2E	**227**	**228**	71,01	0,039	0,364	0,400	0,0060	1,20	1,31	0,24	2,20	3,42
	228	**229**	76,66	0,376	0,590	0,630	0,0060	2,13	0,70	0,24	3,42	4,17
	229	**230**	60,74	0,414	0,590	0,630	0,0060	2,17	0,65	0,22	4,17	4,53
	230	**231**	73,39	0,652	0,775	0,800	0,0060	2,45	0,72	0,35	4,53	2,69
	231	**122**	51,24	0,713	0,775	0,800	0,0060	2,50	0,68	0,35	2,69	1,00
2F	**209**	**225**	81,41	8,655	1,968	2,000	0,0030	3,52	0,25	1,43	4,07	3,22
	225	**226**	84,73	8,639	1,968	2,000	0,0030	3,52	0,25	1,43	3,22	1,50

Table 12. Design collectors results of catchment N3

N3 COLLECTORS' DESGIN												
Collector code			Collectors characteristics									
Segment	Upstream manhole	Downstream manhole	L	Q	Internal diameter	External diameter	Slope	V	Fr	Depth	Upstream covering	Downstream covering
-	-	-	m	m3/s	m	m	-	m/s	-	m	m	m
3A	**310**	**311**	71,14	0,373	0,775	0,800	0,0030	1,65	0,57	0,34	3,40	4,01
	311	**312**	89,87	0,755	0,775	0,800	0,0030	1,90	0,31	0,17	4,01	3,90
	312	**313**	74,45	1,157	0,970	1,000	0,0030	2,16	0,37	0,30	3,90	1,66
	313	**314**	95,5	1,490	1,103	1,200	0,0030	2,31	0,39	0,40	1,66	1,57
	314	**315**	78,83	1,596	1,103	1,200	0,0030	2,34	0,36	0,36	1,57	1,84
	315	**316**	88,44	1,884	1,103	1,200	0,0030	2,40	0,29	0,26	1,84	1,73
	316	**317**	54,33	2,249	1,268	1,300	0,0030	2,55	0,36	0,43	1,73	1,12
	317	**304**	54,33	2,485	1,268	1,300	0,0030	2,60	0,32	0,37	1,12	1,02
3B	**318**	**319**	59,19	0,076	0,364	0,400	0,0040	1,23	0,73	0,19	1,93	2,18
	319	**320**	59,19	0,136	0,452	0,500	0,0040	1,42	0,68	0,19	2,18	1,81
	320	**321**	46,37	0,505	0,775	0,800	0,0040	1,98	0,61	0,35	1,81	1,17
	321	**322**	51,57	0,564	0,775	0,800	0,0040	2,03	0,57	0,35	1,17	1,00
	322	**323**	54,45	0,608	0,775	0,800	0,0040	2,06	0,54	0,31	1,00	1,03
	323	**324**	94,18	0,963	0,970	1,000	0,0040	2,32	0,55	0,41	1,03	1,86
	324	**325**	71,87	1,237	0,970	1,000	0,0040	2,45	0,46	0,34	1,86	2,75
	325	**307**	89,5	1,272	0,970	1,000	0,0040	2,47	0,45	0,34	2,75	3,56
3C	**326**	**327**	78,59	0,328	0,590	0,630	0,0030	1,56	0,40	0,17	1,79	1,25
	327	**328**	91,63	0,591	0,775	0,800	0,0030	1,83	0,43	0,28	1,25	1,00
	328	**329**	83,77	0,831	0,970	1,000	0,0030	2,01	0,48	0,44	1,00	1,92
	329	**330**	83,77	0,886	0,970	1,000	0,0030	2,04	0,46	0,42	1,92	3,15
	330	**309**	86,4	1,498	1,103	1,200	0,0030	2,31	0,38	0,40	3,15	4,36
3H	**345**	**346**	74,28	0,081	0,364	0,400	0,0037	1,21	0,65	0,22	3,40	3,03
	346	**347**	71,65	0,460	0,775	0,800	0,0037	1,88	0,60	0,40	3,03	2,96
	347	**348**	91,07	0,813	0,775	0,800	0,0037	2,10	0,37	0,18	2,96	2,25
	348	**349**	99,07	0,903	0,970	1,000	0,0037	2,22	0,54	0,45	2,25	2,96
	349	**350**	82,16	1,226	0,970	1,000	0,0037	2,37	0,43	0,33	2,96	1,03
3I	**340**	**341**	68,39	0,104	0,364	0,400	0,0035	1,24	0,47	0,28	1,00	1,10
	341	**342**	68,82	0,572	0,775	0,800	0,0035	1,93	0,50	0,31	1,10	3,19
3J	**343**	**344**	56,42	0,071	0,364	0,400	0,0040	1,21	0,76	0,21	2,02	1,40
	344	**342**	56,42	0,127	0,452	0,500	0,0040	1,40	0,70	0,21	1,40	1,00
3K	**342**	**351**	91,36	0,761	0,775	0,800	0,0030	1,90	0,30	0,16	3,20	2,64
	351	**352**	94,53	0,826	0,970	1,000	0,0030	2,01	0,48	0,44	2,64	2,20
	352	**353**	85,17	1,269	0,970	1,000	0,0030	2,19	0,33	0,26	2,20	2,15
	353	**350**	95,76	1,287	0,970	1,000	0,0030	2,19	0,32	0,26	2,15	1,95
3L	**354**	**355**	86,47	0,502	0,775	0,800	0,0030	1,768	0,49	0,3	1,00	1,18
	355	**350**	79,37	1,179	0,970	1,000	0,0030	2,162	0,36	0,3	1,18	1,37
3P	**350**	**356**	97,88	3,626	1,468	1,500	0,0030	2,859	0,31	0,43	2,10	3,24
	356	**357**	86,48	3,765	1,468	1,500	0,0030	2,876	0,29	0,4	3,24	1,43
3O	**359**	**360**	63,42	0,294	0,590	0,630	0,0030	1,534	0,46	0,4	1,00	1,10
	360	**361**	73,7	0,680	0,775	0,800	0,0030	1,876	0,37	0,56	1,10	1,04
	361	**357**	63	1,023	0,970	1,000	0,0030	2,104	0,41	0,61	1,04	1,00
3Q	**357**	**358**	84,39	5,224	1,668	1,700	0,0030	3,125	0,29	0,47	1,50	2,05
	358	**333**	87,89	5,418	1,668	1,700	0,0030	3,142	0,27	0,44	2,05	2,25
3S	**307**	**308**	66,2	15,380	2,468	2,500	0,0028	3,957	0,22	0,6	3,20	3,23
	308	**309**	78,16	15,447	2,468	2,500	0,0028	3,958	0,22	0,59	3,23	3,01

3R	**309**	**331**	89,55	16,613	2,568	2,600	0,0027	3,985	0,22	0,64	3,00	4,22
	331	**332**	65,13	16,827	2,568	2,600	0,0027	3,99	0,21	0,62	4,22	4,00
	332	**333**	68,07	16,848	2,568	2,600	0,0027	3,991	0,21	0,62	4,00	3,85
3T	**333**	**334**	79,86	21,180	2,868	2,900	0,0023	3,971	0,18	0,66	3,55	3,54
	334	**335**	97,46	21,271	2,868	2,900	0,0023	3,972	0,18	0,66	3,54	3,32
	335	**336**	85,64	21,413	2,868	2,900	0,0023	3,974	0,18	0,64	3,32	2,10
3D	**362**	**363**	82,68	0,373	0,775	0,800	0,0030	1,645	0,57	0,4	1,61	1,50
	363	**364**	91,06	0,449	0,775	0,800	0,0030	1,722	0,52	0,4	1,50	1,00
	364	**365**	95,26	0,751	0,775	0,800	0,0030	1,898	0,31	0,61	1,00	1,66
	365	**366**	83,77	1,068	0,970	1,000	0,0030	2,123	0,4	0,62	1,66	2,63
	366	**367**	73,34	1,215	0,970	1,000	0,0030	2,173	0,35	0,69	2,63	3,08
	367	**336**	81,05	1,459	1,103	1,200	0,0030	2,299	0,39	0,4	3,08	3,85
3E	**368**	**369**	87,29	0,094	0,364	0,400	0,0035	1,217	0,54	0,25	1,03	1,05
	369	**370**	98,62	0,362	0,590	0,630	0,0035	1,695	0,42	0,16	1,05	1,66
	370	**371**	97,98	0,689	0,775	0,800	0,0035	2,006	0,43	0,24	1,66	2,42
	371	**338**	99,45	0,921	0,970	1,000	0,0035	2,183	0,51	0,42	2,42	3,14
3M	**226**	**301**	90,86	11,487	2,168	2,200	0,0030	3,763	0,23	0,5	3,05	2,41
	301	**302**	97,73	11,786	2,168	2,200	0,0030	3,77	0,22	0,46	2,41	3,20
	302	**303**	95,75	12,017	2,268	2,300	0,0030	3,844	0,25	0,63	3,20	3,55
	303	**304**	85,58	12,203	2,268	2,300	0,0030	3,852	0,25	0,61	3,55	3,05
3N	**304**	**305**	79,49	14,152	2,368	2,400	0,0030	3,981	0,23	0,59	3,00	3,59
	305	**306**	79,8	14,228	2,368	2,400	0,0030	3,983	0,23	0,58	3,59	3,01
	306	**307**	61,39	14,371	2,368	2,400	0,0030	3,987	0,23	0,57	3,01	2,86
3U	**336**	**337**	56,15	22,531	2,968	3,000	0,0022	3,97	0,18	0,7	2,30	1,81
	337	**338**	56,03	22,554	2,968	3,000	0,0022	3,971	0,18	0,7	1,81	1,50
3V	**338**	**339**	99	23,193	2,968	3,000	0,0022	3,98	0,17	0,64	4,00	3,28
3W	**339**	**386**	90,32	23,220	2,968	3,000	0,0022	3,98	0,17	0,63	3,28	2,06
3F	**372**	**373**	68,9	0,298	0,590	0,630	0,0045	1,806	0,5	0,34	1,75	1,00
	373	**378**	80,42	0,367	0,590	0,630	0,0045	1,887	0,5	0,4	1,00	1,12
3G	**378**	**379**	85,72	1,360	0,970	1,000	0,0030	2,204	0,29	0,22	2,20	1,41
	379	**380**	81,43	1,557	1,103	1,200	0,0030	2,328	0,37	0,38	1,41	2,02
	380	**381**	89,35	1,827	1,103	1,200	0,0030	2,389	0,31	0,28	2,02	1,68
	381	**382**	59,34	1,969	1,103	1,200	0,0030	2,404	0,26	0,22	1,68	1,57
	382	**383**	44,51	2,055	1,268	1,300	0,0030	2,507	0,38	0,48	1,57	1,90
	383	**384**	95,75	2,242	1,268	1,300	0,0030	2,552	0,36	0,44	1,90	2,30
	384	**385**	97,12	2,473	1,268	1,300	0,0030	2,597	0,32	0,37	2,30	3,22
	385	**386**	92,9	2,891	1,368	1,400	0,0030	2,71	0,33	0,44	3,22	4,09
3Z	**374**	**375**	47,39	0,029	0,364	0,400	0,0077	1,205	1,69	0,27	1,75	1,88
	375	**376**	61,99	0,198	0,452	0,500	0,0077	1,993	0,91	0,18	1,88	2,00
	376	**377**	57,23	0,513	0,590	0,630	0,0077	2,496	0,67	0,18	2,00	2,22
	377	**378**	69,74	0,645	0,775	0,800	0,0077	2,687	0,88	0,41	2,22	2,20

From the design of the current collector system, we obtain a peak outflow at the discharge point equivalent to:

$$Q_{p,discharged} = 28,29\ l/s$$

4.3 Alternative 2. Stormwater tank solution

Alternative 2 consists of the previous conventional drainage system, which ends in a storm tank that allows for the lamination and storage of runoff water to reduce the peak flow and control the flow regime towards La Saleta ravine.

4.3.1 Introducing stormwater tanks

Storm tanks function as retention basins that retain part of the volume of the runoff flow hydrograph and reduce peak flows by storage. There are two types of storm tank configurations: non-bypassed and bypassed.

Bypassed storm tanks, also called " in parallel ", are located outside of the collector line, whereby the water is diverted through a relief structure and a conduit. The advantages of this configuration are that they can be located anywhere and that they are able to control environmental pollution by storing the first flush runoff water, which is the most polluted (Tueros, 2000).

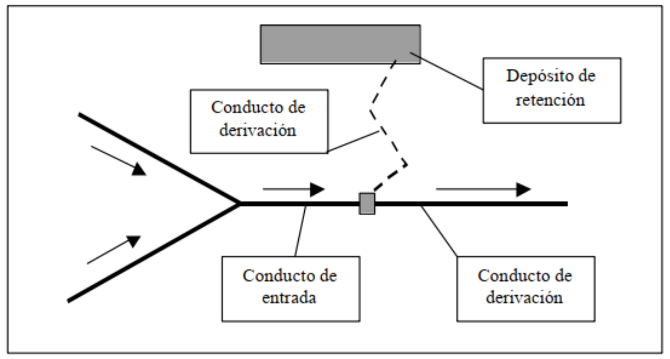


Figure 13. Stormwater tanks "in parallel" scheme. Source: Tueros, 2000

Non-bypassed storm tanks, also called "in series", are located on the collector line so that the entire flow passes through the storm tank and adequately laminates the inflows. In this type of storm tank, the most important hydraulic parameter is the volume, and therefore the plan surface area, to control the water levels in the tank. The advantages of the series tank are that they are simple to design and operate and, in most cases, can be emptied by gravity. However, the disadvantage is that, on occasions, it is not possible to have sufficient surface area to laminate the flows.

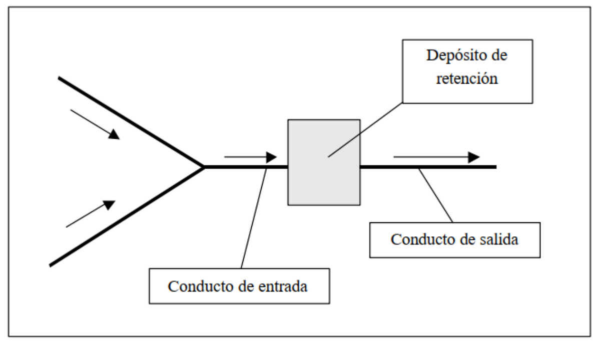


Figure 14. Stormwater tanks "in series" scheme. Source: Tueros, 2000

Because the proposed storm tank is to be underground, there is no real surface area constraint involved. Because of this and the other advantages already mentioned, it has been decided that the storm tank layout of the study will be **in series**.

4.3.2 Layout

The plan layout of this solution follows the same layout of the sewer line sections as in alternative 1, but in this case, we will add a storm tank in line with the last section before the discharge into La Saleta ravine.

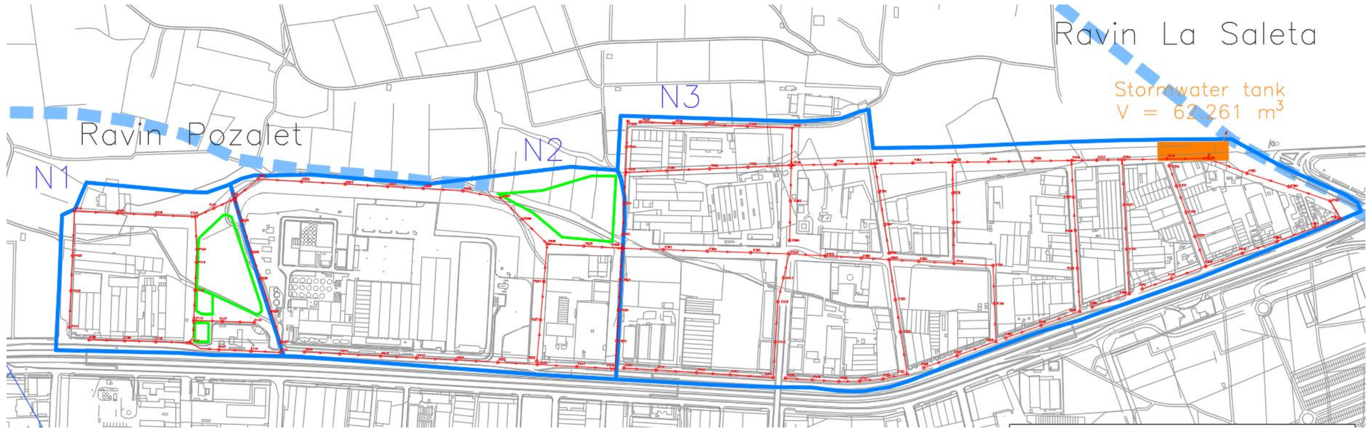


Figure 15. Alternative 2 layout

4.3.3 Collectors design

Since the hydrological and hydraulic calculation is the same for this alternative as for alternative 1, the dimensioning of the collectors will also be the same.

From the design of the current collector network, we obtain a peak outflow at the discharge point equivalent to:

$$Q_{p,discharged} = 28,29\ l/s$$

4.3.4 Stormwater tank design

The design of the storm tank to be implemented in this solution seeks to solve the main problem of the study. This being the discharge into La Saleta ravine with the same flow as in the case of the natural state before the urbanisation.

Therefore, considering that it has been decided to implement a storm tank system in series, it must be designed with dimensions that manage to store enough water volume so that, at the outlet of the storm tank, the peak flow at any given time is the same or less than the flow in the natural state.

To implement this solution, the storm tank will be designed to approximate the optimum dimensions and thus achieve its objective based on the calculations made for the conventional collector system. Subsequently, the storm tank will be modelled in SWMM to adjust the design dimensions and meet the target for the design storms.

The design is based on data on the volume to be stored and the outlet flow to be achieved. There are different numerical studies to determine the design, in this study it has been determined that the outlet should be free and flooded and that the spillway should be of the weir type.

To understand how the sizing of the storm tank is carried out, it is necessary to look at the hydrogram that represents the inflow over time. This hydrogram identifies the maximum flow rate at a given time. To ensure that the outflow is equal to the natural flow, there is a tendency to erroneously think that the storm tank must be sized to store all the water corresponding to the flows that exceed the value of the natural flow. This flow rate, which is justified in the annex of initial data, corresponds to:

$$Q_{p,nat} = 6,88\ l/s$$

This statement is considered erroneous because it does not take into account the behaviour of the outflow hydrograph. Thus, in a series storm tank, where the outflow hydrograph is shifted to the right and its peak flow decreases, it would be represented as follows:

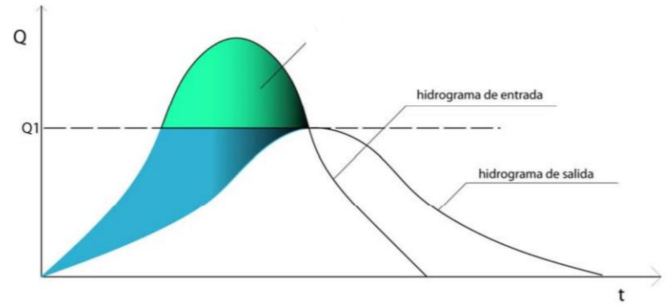


Figure 16. Example of volume for stormtank design in series. Source: M. Gómez, 2004

In order for the outflow hydrograph to really have the target flow rate represented by a broken line as the peak flow rate, not only should the water corresponding to all the flows that exceed this threshold peak flow rate, represented in the figure in green, be stored, but also all the water that, before reaching the peak flow rate, is located between the inflow and outflow hydrograph, represented in the figure in blue.

In parallel storm tanks, the above statement is not so erroneous due to the behaviour of the outflow hydrographs in parallel tanks.

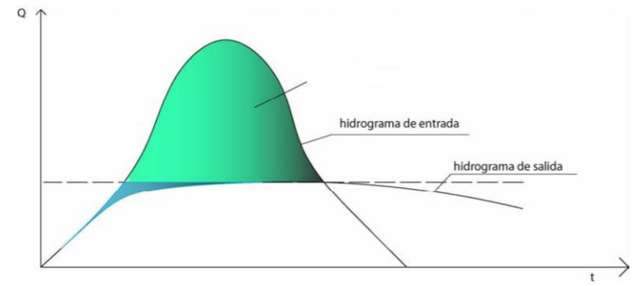


Figure 17. Example of volume for stormtank design in parallel. Source: M. Gómez, 2004

For this first design, which will serve as the basis for the modelling in SWMM, the synthetic unit hydrograph will be represented from the data obtained from the calculation of the collector network in the last section and in this way determine the volume required for storage.

The synthetic unit hydrograph is carried out using the triangular hydrograph method of the Soil Conservation Service (SCS) of the United States. This method proposes a synthetic hydrograph with a triangular shape whose maximum value is the peak flow and whose duration corresponds to times related to the basin concentration time.

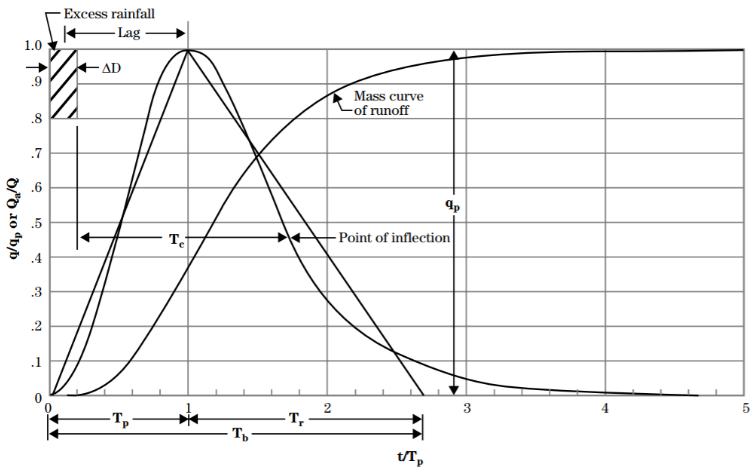


Figure 18. Unit triangular hydrogram scheme of SCS. Source: National Resources Conservation Service, 2007

Where:
- tc (min), is the time of concentration in the catchment.
- tp (min), is the peak time, considered from the start to reach the peak flow of the triangular hydrogram.
- tb (min), is the base time of the triangular hydrogram.
- tr (min) is the delay time from the tp to the end of the triangular hydrogram.

The time of concentration for each line segment of the collector network was calculated according to the calibrated rational method indicated before. Specifically, for the final segment, it corresponds to:

$$tc = 26,3\ min$$

Table 13. Rational calibrated method results for final segment

Segment	Upstream manhole	Downstream manhole	ts	tc	L	V	L/V	I
-	-	-	min	min	m	m/s	s	mm/h
3W	**339**	**386**	6	26,3	90,32	3,98	1015,4	99,9

The peak time can be obtained from knowledge of the time of concentration as follows:

$$tp = \sqrt{tc} + 0,6 * tc$$

Also, the base time is known from the peak time:

$$tb = 2,67 * tp$$

By means of these equations we have been able to determine all the representative times of the hydrograph for the final segment:

Table 14. SCS triangular hydrograph times of the final segment

Segment	Upstream manhole	Downstream manhole	tc	tp	tb	tr
-	-	-	min	min	min	min
3W	**339**	**386**	26,3	20,9	55,8	34,9

With the times and flows representative of the last segment of the collector system we can represent the triangular hydrogram of the final segment of the collector system and calculate the required storage area:

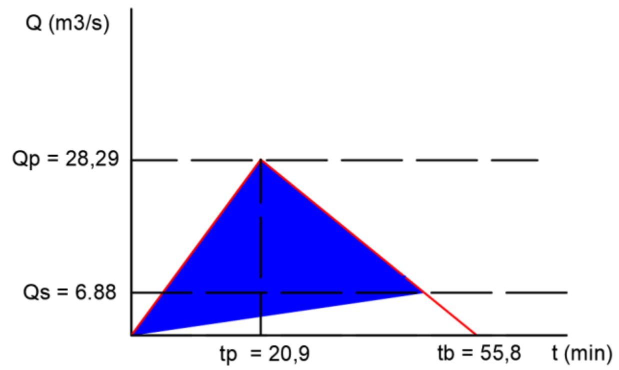


Figure 19. Triangular hydrogram representing stormtank storage needs

As can be seen from the graph, an attempt has been made to approximate what would correspond to the storage volume required in a series storm tank by including the volume that lies between the line corresponding to the threshold flow and an inclined line from the beginning of the triangular hydrograph to the line of intersection of the hydrograph with the threshold flow.

Thus, we obtain that the volume of water to be stored, and therefore the design volume of the storm tank, will be:

$$V_{tank} = \mathbf{36.029\ m3}$$

Among the different typologies that could be used for the storm tank to be designed, a reinforced concrete storm tank with a rectangular shape and installed underground has been chosen, as this saves space on the surface, the rectangular shape makes it easy to implement automatic cleaning systems and it does not have the disadvantages of the other typologies.

Figure 20. Example of an underground storm tank in an urban area. Source: MAGRAMA, 2014

Figure 21. Example of an underground storm tank: Arroyo Fresno tank (Madrid). Source: Canal Isabel II, 2019

The dimensions of the storm tank obtained from the calculation of its volume shall be:

$$S_{tank} = \mathbf{5.160\ m2}$$
$$h_{tank} = \mathbf{7,0\ m}$$

Once the dimensions of the storm tank are known, its spillway will be dimensioned, this being of the weir type. The dimensions are obtained from the outlet flow:

$$Q = Kw * L * h^{3/2}$$

Where:
- Q (m3/s) is the outlet flow rate
- Kw (adimensional) is the discharge coefficient of the weir
- L(m) is the length of the weir
- h (m) is the loading height

A spillway discharge coefficient of 1,1 is considered (Gómez, 2012) and the rest of the parameters are obtained from the natural flow, being:

$$h = 0,25\ m$$
$$L = 50,1\ m$$

4.3.5 Stormwater emptying system

When considering the design of a storm tank, the method of emptying the storm tank after the peak of the rainfall event must be identified. Up to now the entire drainage system has been gravity based, as it has been designed for this purpose. Therefore, the collection of storm water is done without the need for a pumping station, which adds costs, not only during construction, but throughout the life of the system. However, when a buried storm tank is proposed in the solution, it should be considered whether the emptying of the tank requires a pumping system.

The proposal for emptying will be such that the stored water will be discharged, once the rainfall event has passed, into the separate sewage collection network. Information on the location of the sewage networks is provided by the Entidad Publica de Saneamiento de Aguas Residuales de la Comunidad Valenciana (EPSAR) [12], which locates the nearest sewage network to the project area on the highway adjacent to the study area, the A3. This sewage network is connected to the Quart-Benager wastewater treatment and purification plant, with a design treatment flow of 60,000 m3/day, which as of 2019 is treating 31,733 m3/day.

This conveyance scheme is chosen because the water stored in the tank has a higher concentration of contamination than the rest of the water discharged to the receiving environment because it is the first water in the runoff, the "first flush". Connecting the stored water from the tank would allow this water to be treated at times when the station is operating at low capacity.

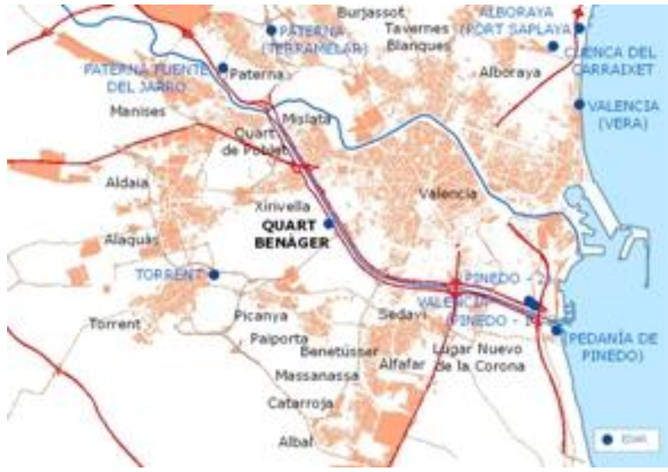


Figure 22. Collectors and WWTP network. Source: EPSAR, 2019

The collector network passing through the A3 is located at a depth of 2 metres. To check whether the tank can be emptied by gravity, the lower level of the tank should be compared with the level of the collector network that runs in the direction of the Quart-Benager WWTP. If the collector level is below the tank level, the tank can be emptied by gravity. If this is not the case, a pumping system is required to lift the water up to the level of the sewage network.

$$z,inf,tank = +46,64\ m$$

$$z,sewage = +60,50\ m$$

As the level of the lower part of the tank is below, the stored water must have a pumping system that raises it by 13.96 m.

The characteristics of the pump to be implemented are calculated from the definition of the emptying flow rate and the manometric head.

The flow rate chosen is the one that can empty the entire storm tank in half a day.

$$V_{tank} = \mathbf{36.029\ m3}$$
$$t_{empty} = \mathbf{12\ h}$$

$$Q_{empty} = \mathbf{834\ l/s}$$

The manometric height between two points, A and B, is calculated by means of the Bernoulli equation:

$$H = (z_B - z_A) + \left(\frac{P_B}{\gamma} - \frac{P_A}{\gamma}\right) + \left(\frac{V_B{}^2}{2g} - \frac{V_A{}^2}{2g}\right) + \sum \frac{8 * f * L * Q^2}{g * \pi^2 * D^5} + \sum \frac{8 * k * Q^2}{g * \pi^2 * D^4}$$

In this rough calculation, friction losses and singular losses have been excluded.

$$H = \mathbf{13{,}96\ m}$$

After defining the flow rate and manometric height that the pump must supply, it is necessary to go to pump manufacturers who have a quick selection table that allows us to obtain the model that offers the best performance from the whole range of pumps they offer.

The following figure shows a selection table obtained from the pump manufacturer's catalogue " Bombas ideal", which shows the zones with the best performance for each pump model within the magnitudes of our pumping system:

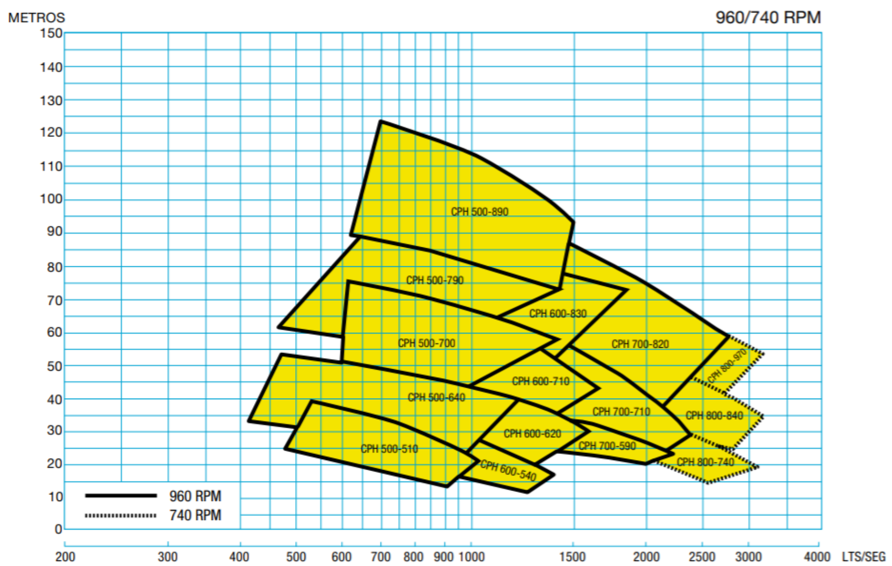


Figure 23. Pump choice considering flow and manometric height. Source: Bombas ideal

From the table above, the pump that corresponds to the characteristics of the pumping system is a pump type **CPH 500-510**, 50 Hz and 960 RPM. In more detail, the manufacturer provides a graph in which its characteristics related to diameter and efficiency can be determined.

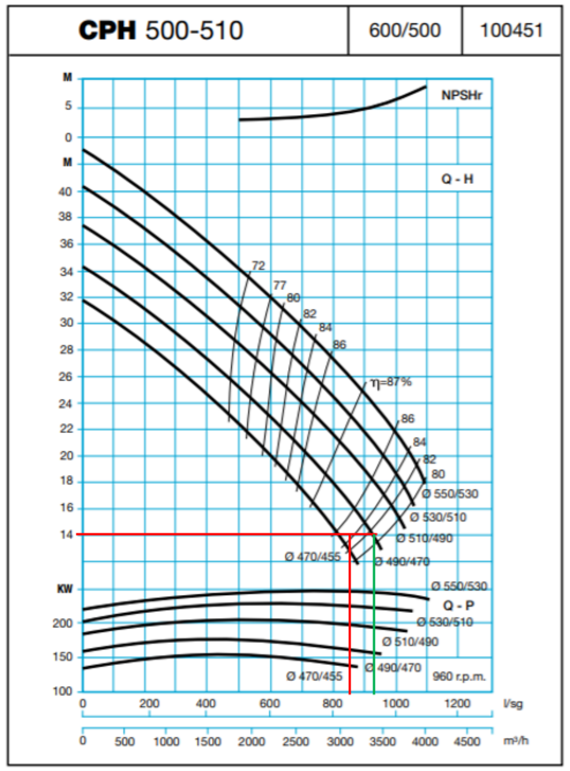


Figure 24.. Specification pump choices on CPH500-510 pump. Source: Bombas ideal

In red is identified the point at which our pumping needs are located, between 470/455 and 490/470 diameters. The pump selection should be the one immediately above, since the 470/455 diameter would be too small for the design flow. In green, the exact point that the pump we have chosen has been identified. This gives an efficiency and pump diameter corresponding to:

$$\boldsymbol{\eta = 81\%}$$
$$\boldsymbol{\emptyset = 490/470}$$

And a real discharge flow rate equal to:

$$\boldsymbol{Q, empty = 930\ l/s}$$

This will allow the tank to be emptied in **10,8 hours**.

4.4 Alternative 3. SuDS solution

The Territorial Action Plan of the Comunidad Valenciana (PATRICOVA, 2015) states that efforts will be made to develop the use of SuDS in the design of drainage systems. It reads as follows. Therefore, this alternative seeks to implement a solution that incorporates sustainable drainage systems. This section will identify the sustainable drainage systems used, justifying their emplacement. As well as type sections, parameters and processes involved. It reads as follows:

"El drenaje de las aguas pluviales en las áreas urbanas de superficie mayor a veinte hectáreas (20 ha) cumplirá las siguientes condiciones […] Se fomentará el uso de Sistemas Urbanos de Drenaje Sostenible en todos los municipios de la Comunitat Valenciana."

Therefore, this alternative seeks to implement a solution that incorporates sustainable drainage systems. This section will identify the sustainable drainage systems used, justifying their emplacement. As well as type sections, parameters and processes involved.

4.4.1 Suitability of SuDS typology to the study area

In *Annex Nº1: Technical context. Sustainable Drainage Systems*, the characteristics of the different types of structural SUDS and their suitability for industrial areas have been defined. This section, as part of the proposal of solutions, is based on the previously mentioned to determine which SUDS are best suited to the Quart de Poblet industrial estate.

To this end, the suitability of the typologies in industrial areas in general and the requirements of the location of the study area are considered.

4.4.1.1 Infiltration feasibility

Soil infiltration is a necessary process in sustainable drainage systems with infiltration as its own function since the design of the drainage system's emptying and overflow depends on it.

Therefore, to ensure that sustainable drainage systems with infiltration are feasible in the study area, the infiltration of the soil itself must first be ensured. This will occur if the following criteria are met:

- **Permeability coefficient greater than 10^{-6} m/s**. This criterion is indicated to allow the soil to absorb water at a rate that allows the sustainable management systems to be emptied in less than 48 hours.
- **Distance from the water table greater than 1 metre**. This criterion is indicated for three reasons, the first is that a shorter distance would not allow the infiltration of unsaturated soil, the second is that there would be a high risk of contamination if the water table is hydraulically linked to the collected water and finally because it could lead to a decrease in the storage capacity of the SuDS.
- **Distance to foundations greater than 3 metres**. The flow through the ground could cause instability of the foundations if it is located very close and could cause the foundations to be displaced.

4.4.1.2 Terrain slopes

In some cases, the slopes of the terrain may limit the selection of SuDS. The following table shows the recommended slopes for each type of structural SuDS.

Table 15. Recommended maximal slopes. Source: Ayuntamiento de Madrid, 2018

SuDS typology	Slope (%)
Trees	-
Infiltration trenches	2 - 6
Reticular tanks	< 15
Pervious pavements	< 3
Filter drains	< 2
Rain gardens	< 10
Bioretention systems	< 10
Swales	0,5 - 6
Wetlands	< 10
Ponds	< 10
Detention basins	< 10
Infiltration basins	< 10
Green roofs	< 25
Attenuation storage tanks	-

4.4.1.3 SuDS footprint

The space occupied by SuDS can be determined by the optimum of the ratio of the floor area to the impervious area of the catchment. The following table also shows the recommendations of the Madrid townhall (Ayuntamiento de Madid, 2018) for this ratio.

Table 16. Recommended footprint. Source: Ayuntamiento de Madrid, 2018

SuDS typology	ASuDS/Aimp (%)
Trees	-
Infiltration trenches	5 - 10
Reticular tanks	Not applicable
Pervious pavements	33 - 100
Filter drains	5 - 10
Rain gardens	3 - 30
Bioretention systems	3 - 30
Swales	0,5 - 6

Wetlands	3 - 30
Ponds	3 - 30
Detention basins	3 - 30
Infiltration basins	3 - 30
Green roofs	50 - 80
Attenuation storage tanks	-

4.4.1.4 Pollutant assessment

The choice of SuDS is conditional on the assessment of pollutants in the area, especially in the study area where the pollution hazard indexes are high.

Mention has already been made in the technical context on the method of pollution risk level index and how the mitigation of pollutants by SuDS must be sufficient to counteract the pollution hazard index of the industrial area. In the following, the most relevant for the choice of SuDS in the study area is extracted, which corresponds to the pollution hazard levels of industrial areas and the mitigation levels of the different SuDS.

Table 17. Pollution hazard levels at industrial states. Source: Woods Ballard, B. et al., 2015

Total Suspended Solids	Metals	Hydrocarbons
0,8	0,8	0,9

Table 18. Mitigation indicators for SUDS in surface water discharges. Source: Perales, S. et al, 2019

SuDS typology	Total Suspended Solids	Metals	Hydrocarbons
Trees	0,6	0,5	0,6
Infiltration trenches	0,4	0,4	0,4
Reticular tanks	-	-	-
Pervious pavements	0,7	0,6	0,7
Filter drains	0,4	0,4	0,4
Rain gardens	0,6	0,5	0,6
Bioretention systems	0,8	0,8	0,8
Swales	0,6	0,6	0,6
Wetlands	0,8	0,8	0,8
Ponds	0,7	0,7	0,5
Detention basins	0,5	0,5	0,6
Infiltration basins	0,6	0,5	0,6
Green roofs	0,4	0,4	0,4
Attenuation storage tanks	-	-	-

4.4.2 SuDS typology proposal

Considering all the elements previously mentioned both in this annex and *in Annex Nº1: Technical framework. Sustainable Drainage Systems*, this section sets out the sustainable drainage systems chosen and their justification.

Of all the above, the parameters that least limit the choice of SUDS in our specific case are:

- **Permeability coefficient**. The permeability coefficient is $3.58 \cdot 10^{-6}$ m/s, which is three times higher than the limit for the viability of infiltration. Therefore, this parameter is not limiting.
- **Water level.** The water level in the study area is well below one metre from the ground surface, so the water table does not limit infiltration.
- **Foundation distance.** In order to protect the foundations of the buildings, a three-metre wide perimeter of the residential plots will be considered, which will be taken into account when developing the solution but does not limit the typology.
- **Maximum slopes.** In the slope plan of the study area, it can be seen that the average slopes of the sections are, in all cases, less than 3%. Although there are certain areas with slopes of more than 2%, the use of filter drains will be discarded. However, if at any time the slopes are too steep for the SUDS, the elements can always be compartmentalised. This compartmentalisation will be explained later.

In this area, it is determined that the most limiting factor in the selection of SUDS is the mitigation indexes. A combination of SUDS must be implemented that achieve a mitigation index higher than the pollution hazard index of industrial areas with respect to suspended solids, metals and hydrocarbons, which are 0.8, 0.8 and 0.9 respectively.

Considering that no single typology is able to mitigate the totality of the hazard indexes, a chain should be formed in series according to the following criteria:

$$Total\ mitigation\ index = Mitigation\ index\ 1 + 0{,}5 * mitigation\ index\ n$$

First, **pervious pavement** is chosen as the first element of the chain along the streets of the three study catchment areas. This pervious pavement will be installed as parallel parking along the streets whose width allows it. The parking will be reserved for cars only, although the design will consider the fact that heavier vehicles will occasionally generate stresses on the pervious pavement.

Table 19. Mitigation index for pervious pavement. Source: Perales, S. et al, 2019

SuDS typology	Total Suspended Solids	Metals	Hydrocarbons
Pervious pavements	0,7	0,6	0,7

It should be noted that as the first element in the chain and the runoff water must pass through a second system to achieve the necessary mitigation rates, these pervious pavements will have a geotextile that will prevent water infiltration into the ground.

The Law of Territorial Planning, Urbanism and Landscape of the Comunidad Valenciana (LOTUP) indicates that car parks must have a minimum dimension of 2.80 x 2.20 m, so it has been decided that the width of these pavements will be 3 metres.

For the implementation of pervious pavements, the total available width of the street must be considered. This will determine whether a side band of pervious pavement parking is allowed, two side bands of parking or, if the width is large enough, the addition of a central median to function as a rain garden.

Considering the available green areas, the implementation of bioretention areas, rain gardens, artificial wetlands and detention and infiltration ponds and basins has been considered. Analysing the sector, it is possible to identify two large green areas in the N1 and N2 catchment that would allow the implementation of these SuDS where the available surface area is limiting. For these large areas, analysing the mitigation of pollutants and allowing the water to infiltrate, it has been decided to place **infiltration basins** due to their high mitigation of pollutants. Ideally, a bioretention area would be available which acts more effectively against pollutants, but the surfaces are too large for a bioretention area to be able to mitigate pollution as effectively

Table 20. Mitigation index for infiltration basins. Source: Perales, S. et al, 2019

SuDS typology	Total Suspended Solids	Metals	Hydrocarbons
Infiltration basins	0,6	0,5	0,6

In this way the main chain of catchments N1 and N2 will try to manage all the runoff water passing first through the pervious pavements and then through the infiltration area placed in the green area of each of the catchments. The case of catchment N3 is more complex, because it does not have an extensive green area in which to discharge the water coming from the pervious pavements. Therefore, as far as possible, the drainage system will be designed so that all water coming from the pervious pavements will pass through the **rain gardens** that will be installed in the centre of the streets. Rain gardens function like bioretention areas only their filter medium is less thick and therefore have a lower mitigation rate than bioretention areas.

Table 21. Mitigation index for rain gardens. Source: Perales, S. et al, 2019

SuDS typology	Total Suspended Solids	Metals	Hydrocarbons
Rain gardens	0,6	0,5	0,6

With the above explanation it is concluded that, in the study area, the mitigation index of the two possible chains of sustainable drainage systems will be:

Table 22. Total mitigation index at the study area

SST	Metales	Hidrocarburos
1	0,85	1

Which are higher, in any of the three pollutants listed, than the pollution hazard index corresponding to industrial areas.

Finally, in the following figure we can see graphically the surface area available to accommodate infiltration SUDS:

Figure 25. SuDS implementation limit by infiltration

As can be seen, this perimeter is not very limiting as it does not affect the green areas of the study area and the permeable pavements that are going to be installed on the road will not be placed next to the buildings; a footpath of at least 3 metres will be placed between the car park and the building.

4.4.3 Alternative 3 layout

The layout of the alternative 3 including all the SuDS structural elements is presented in the figure below:

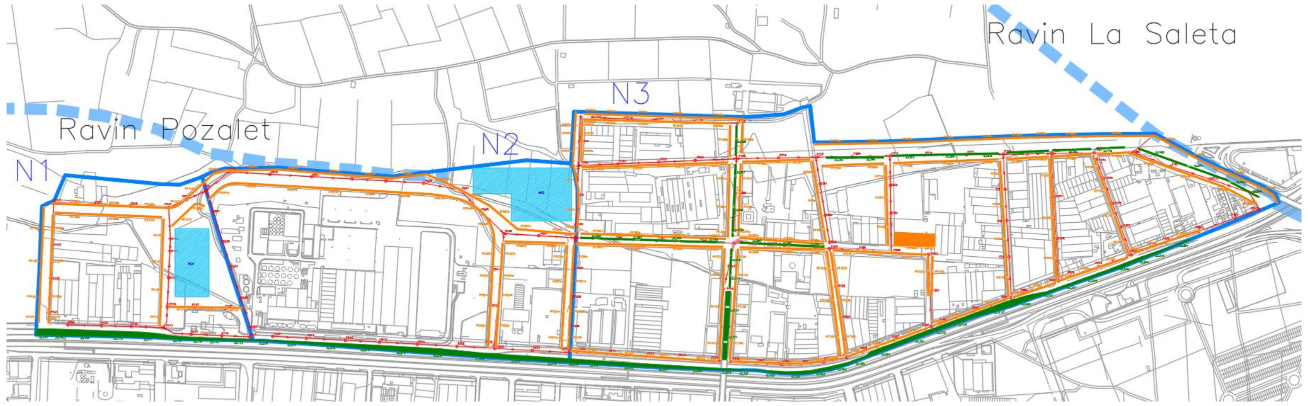


Figure 26. Alternative 3 layout

4.4.4 SuDS cross-sections

Once the types of SUDS to be installed in the study area are known, the cross-section of each type is determined.

4.4.4.1 Pervious pavement

The SuDS manual (Woods-Ballard et al., 2015) classifies the pervious pavements according to whether they have total infiltration, partial infiltration or no infiltration. The selection of one or the other modality, as well as the thickness of each of the layers that will make up these SUDS, depends on the traffic that these surfaces must support and the frequency with which they will do so, as well as whether or not infiltration into the ground is finally decided upon or not.

The Pervious Pavements installed in the area are pavements dedicated to lateral car parks that head the SUDS chain to treat water quality and eliminate pollutants. Therefore, the type of permeable pavements chosen are pavements that do not allow infiltration but are redirected by drains to another SUDS.

Different guidelines are used to determine the cross-section of permeable pavements. The chosen cross-section is as follows:

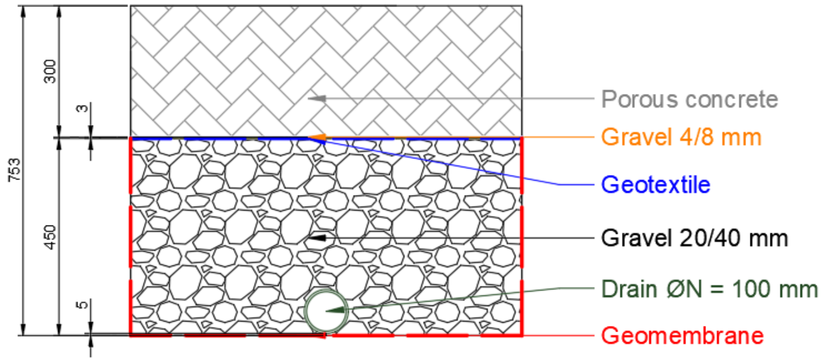


Figure 27. Cross-section on pervious pavement

It has been considered that there is no berm and therefore water cannot pond above the surface before overflow occurs.

For the layer thicknesses, the traffic classification provided by the SuDS manual (Woods-Ballard et al., 2015) has been followed, in which the traffic in the area would be category 6.

Traffic category (BS 7533)	Standard axles per day	Lifetime traffic (msa)	NRSWA road type	Maximum anticipated axle load (kg)	Example number of commercial vehicles per day[1]	Typical application
11	Areas with axle loads greater than permitted by the Road Vehicles (Construction and Use) Regulations 1986 as amended are not included in this document					
10	≤ 4,000	≤ 60	0	Site specific (see Knapton, 2007)		Adopted highways and commercial/industrial developments used by a high number of commercial vehicles Ports and airport landside Bus stops and bus lanes
9	≤ 2,000	≤ 30	1	Site specific (see Knapton, 2007)		
8	≤ 700	< 10	2	8000	Approx 420	
7	≤ 275	< 2.5	3	8000	Approx 170	
6	≤ 60	< 0.5	4	8000	Approx 35	Adopted highways and other roads used by a moderate number of commercial vehicles Pedestrian areas subjected to regular overrun of commercial vehicles Industrial premises Petrol station forecourts

Figure 28. Traffic loading categories for pervious pavement design. Source: Woods Ballard et al., 2015

Although a porous concrete surface course thickness of between 125 and 150 mm is generally recommended, due to the heavy traffic load of the industrial area, the SUDS manual recommends increasing the thickness of the surface course to between 200 - 300 mm. Therefore, it has been determined that the pavement will consist of porous concrete with a layer **thickness of 300 mm** and a void ratio of the porous mix between 0.20 and 0.35 (Lafarge, 2013). A **void ratio n=0.25** has been chosen.

The permeability of new porous concrete is high, but it should be noted that over time this permeability may decrease due to clogging by fine particles carried by runoff. Due to this problem, a clogging factor is applied, which translates into a safety coefficient that should be at least of the order of 10 (Rodríguez Hernández, J., 2008).

Studies have determined that porous concrete can have a permeability between 0.5 and 5 cm/s (Aguado, 1995), we consider that a permeability of 2500 mm/h with a clogging factor of 10 is admissible. In addition, other papers have been found that recommend a minimum permeability of **250 mm/h** (Woods-Ballard et al., 2015), this being on the higher side of safety.

The storage layer has a height between 150 and 450 mm and a void ratio between 0.5 and 0.75 (Woods-Ballard et al., 2015). **A 450 mm thick layer** consisting of a 20/40 gravel layer containing a **void ratio n=0.5** was chosen (Sañudo-Fontaneda et al., 2014).

Between the porous concrete and the storage layer there will be a geotextile that will act as a filter and separator to prevent the transfer of fines and the loss of support at the base.

An impermeable geomembrane will be provided under the storage layer to prevent infiltration into the ground, as all runoff water must pass through another SUDS in the chain before infiltrating into the ground in order to meet the restrictions on contamination rates in industrial areas.

The water will be drained from the permeable pavement through a drainage pipe drainage system that will be buried in the storage layer. The drainage of the drain will follow an orifice type law where the outflow rate is determined by:

$$q = C \cdot (h - Hd)^n$$

Where:
- q (mm /h) is the outflow.
- h (mm) is the stored water height
- Hd (mm) is the height of drainage
- n (adimensional) is the drainage exponent

To make the drain act as an orifice, the typical value is n=0.5 called the drainage exponent. Knowing the value of the drainage exponent, an approximate value for the drainage coefficient can be estimated based on the time t needed to drain to a depth D:

$$C = \frac{2 * D^{0,5}}{t}$$

Where:
- D (mm) is the depth of the drain.
- t (h) is the emptying time.

Assuming an emptying time of 24h, a drainage coefficient of **C = 2.12** is obtained.

A height of 5mm shall be provided between the base of the storage layer and the base of the drain.

4.4.4.2 Rain gardens

Coinciding with the case of permeable pavements, rain gardens can have a configuration of infiltration in the ground or without infiltration and drainage. The choice of infiltration or non-infiltration configuration depends on where in the chain the SUDS is located.

In order to meet the criteria for pollutant indexes, rain gardens will be with infiltration as long as they have already passed through another SUDS component previously, this is the case for certain rain gardens located in N3 catchments. However, all rain gardens located in the N1 and N2 catchments are at the head of the SUDS chain so in this case the runoff water must be drained away and infiltration will not be allowed.

The rain garden cross-section chosen is as follows:

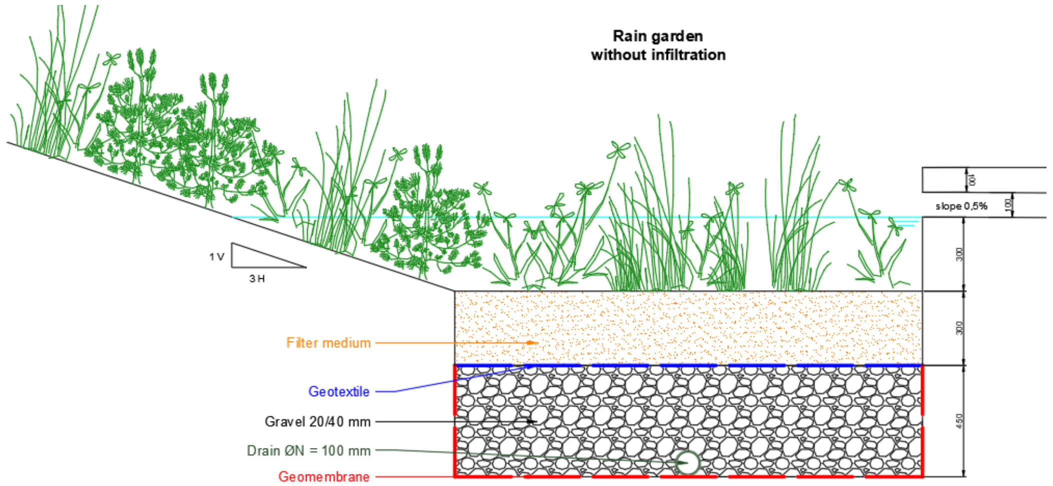


Figure 29. Cross-section on rain gardens without infiltration

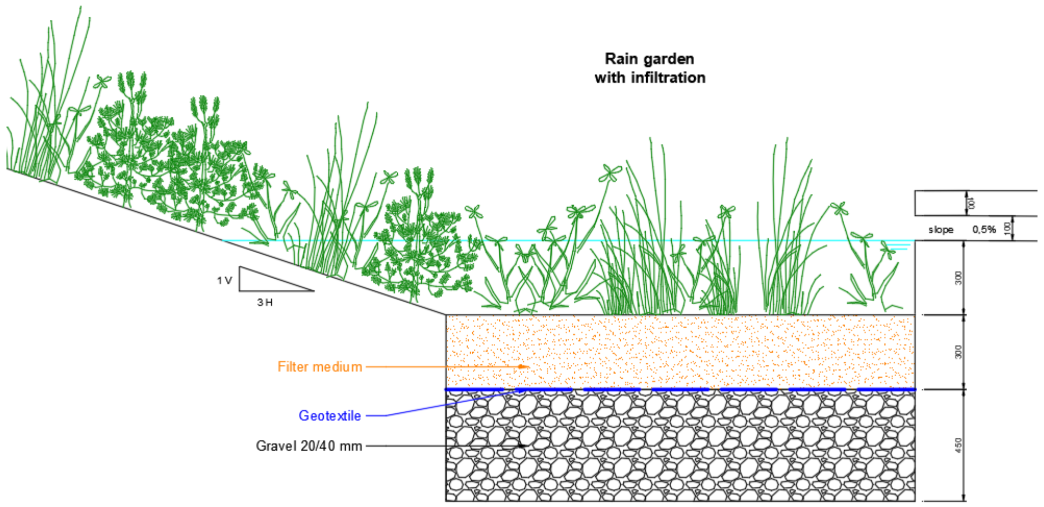


Figure 30. Cross-section on rain gardens with infiltration

In this case, a berm is provided to allow water to pond above the surface before overflow occurs. The surface water storage may be as high as 150 to 300 mm. It has been decided to determine a height of 300 mm of possible surface water storage with a porosity of n=1, because there is no material in this storage.

The soil layer called filter medium will be composed of some source of organic matter and slow release plant nutrients to maintain healthy plant growth. The soil and storage layer together should be between 750 and 1000 mm thick. Therefore, a soil layer as filter medium of **300 mm** with porosity n=0.5 and a storage layer composed of gravels of **450 mm** with void ratio n=0.5 have been determined. The hydraulic conductivity of the soil should be between 100 and 300 mm/h (Woods-Ballard et al., 2015). A saturated hydraulic conductivity of the sand has been estimated at **250 mm/h**.

In case infiltration is allowed the storage layer will have an adjacent soil permeability of **12.9 mm/h** as already indicated in previous sections.

A geotextile shall be provided between the soil layer and the storage layer. Under the storage layer an impermeable geomembrane shall be provided to prevent infiltration into the ground when the rain garden is first in the SUDS chain.

If infiltration is not allowed, the water will be evacuated by a drainage pipe drainage system embedded in the storage layer, as with permeable pavements. In this case, the drainage depth is different and by means of the equations indicated above, a drainage coefficient of **C = 2.28** is obtained and the law of emptying by orifice with a drainage exponent **n = 0.5** is maintained. There will also be a height of 5 mm between the base of the storage layer and the base of the drain.

4.4.4.3 Infiltration basin

Infiltration basins have a standard section that resembles rain gardens, except that infiltration basins function as ponds when there are large episodes of rainfall, allowing larger quantities of water to be stored, both because of their large surface area and the possibility of storing a greater height.

Furthermore, as these basins are the last element in the SUDS chain, they must have a configuration with infiltration into the ground.

In the case of infiltration basins, the berm that allows water to stagnate above the surface before overflow occurs will have a height of **600 mm** with a porosity of **n = 1**.

The rest of the parameters are the same as for the rain gardens. The soil layer is **300 mm** with porosity **n = 0.5** and the gravel storage layer is 450 mm with void ratio **n = 0.5**. The hydraulic conductivity of the soil corresponding to the saturated hydraulic conductivity of the sand is **250 mm/h**. A geotextile is placed between the soil layer and the storage layer.

The resulting cross-section is as follows:

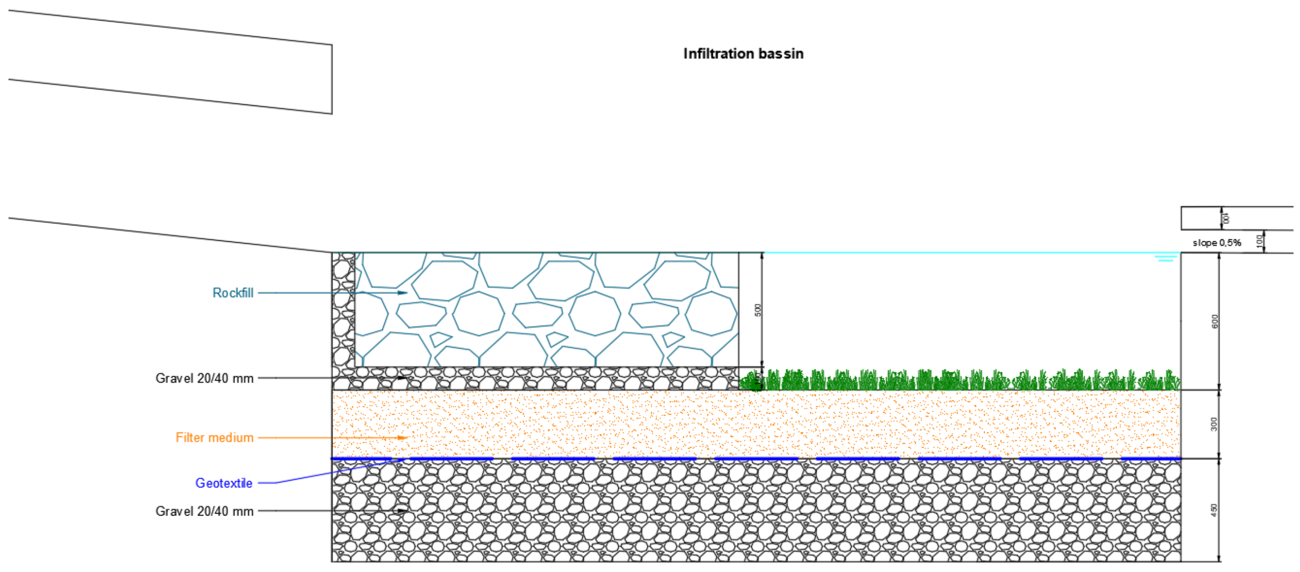


Figure 31. Cross-section of infiltration basin

4.4.5 Volumetric efficiency

Once the standard SuDS sections have been determined, we are going to determine the volumetric efficiency of each of the elements of the drainage system to check that the proposed SuDS can store the volume needed to meet the quantity and quality criteria.

As indicated at the beginning of this annex on the management of volumes to meet design criteria, the aim is to store the volume V_{80} to meet the quantity criterion and V_{90} to meet the quality criterion. However, it is important to note that, according to international references, filtering the entire V_{80} volume of the design rainfall can be considered equivalent to adequately managing the V_{90}, thus fulfilling the quality criterion imposed. Therefore, as will be seen below, it will be accepted in certain cases that the V_{90} volume is not met if the V_{80} is met and, in any case, V_{90} should be managed whenever possible.

The procedure for calculating volumetric efficiency is as follows:

- Grouping the system elements into type sections.
- Studying the potential sites for implementation
- Determination of SUDS surfaces
- Calculation of the volume of runoff generated by the design rainfall under the quantity and quality criteria (V_{80}, V_{90})
- Calculation of net storage of the designed SUDS
- Verification of compliance with design criteria

In the case of SuDS on very sloping surfaces, the space where the water is retained will be compartmentalized by means of transverse flow deflectors that allow the water to be slowed down and temporarily held for future drainage.

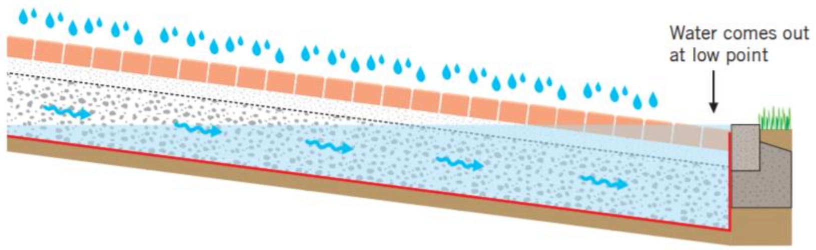


Figure 32. Excessive slope problems. Source: Interpave, 2020

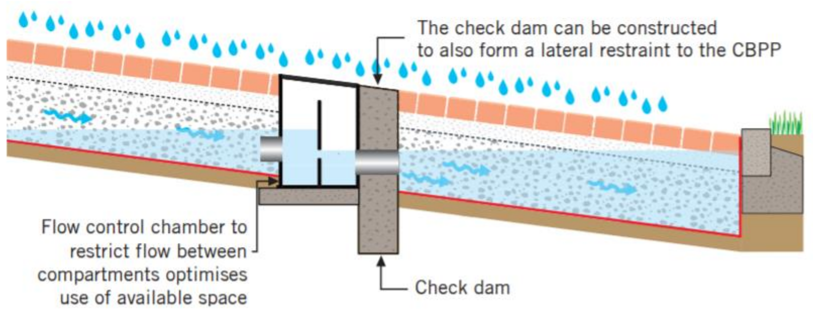


Figure 33. Example of transversal deflectors. Source: Interpave, 2020

In addition, in those SuDS where infiltration is allowed, terraces will be implemented at the base of the storage layer to provide the time lag necessary for percolation to occur properly into the ground.

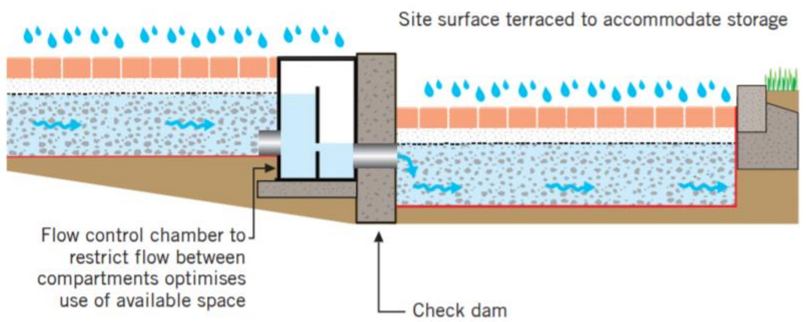


Figure 34. Example of terrace division. Source: Interpave, 2020

4.4.5.1 Volume of produced runoff

The volume of produced runoff for both quality and quantity criteria is calculated as follows:

$$V_{E,80} = \sum_{1=1}^{n} (A_i * C_i) * \frac{V_{80}}{1000}$$

$$V_{E,90} = \sum_{1=1}^{n} (A_i * C_i) * \frac{V_{90}}{1000}$$

where,

- Ai (m2) is the area of the sub-catchment.
- Ci (adimensional) is the runoff coefficient of the sub-catchment.
- V80 (mm) is the 80% percentile rainfall volume to ensure quantity management.
- V90 (mm) is the 90% percentile rainfall volume to ensure management of quality.

The sum of Ai and Ci is the impervious area of the catchment and the following runoff coefficients have been considered to determine it:

Table 23. Runoff coefficients for the study area

Surface	Runoff coefficient
Road	0,95
Private catchment	0,75
Green area	0,30
Pervious pavement	0,70
Infiltration basin	0,30
Rain garden	0,30

4.4.5.2 Storage volume

The storage volume of the SuDS is calculated considering the slope. In the case where compartmentalization by terraces is implemented, where the base of the sections is flat, the volume of SuDS is calculated as follows:

$$V_{SUDS} = L_{alm} * h_{max} * b * n * N$$

Where:

- hmax (m) is the maximum height of the material to be stored in the SuDS.
- b (m) is the width of the area for SuDS.
- n (adimensional) is the void ratio of the storage material.
- N (adimensional) is the number of SuDS sections.

In the general case, where there are no terraces and the base has a slope, the volume of SuDS shall be calculated as follows:

First, the length of the water sheet is calculated for the slope and maximum storage height of each SuDS:

$$L_{wl} = \frac{h_{max}}{pte}$$

In this way for a section of SuDS, with a storage length (L,alm) as depicted in the picture below, two volume calculation methodologies are possible.

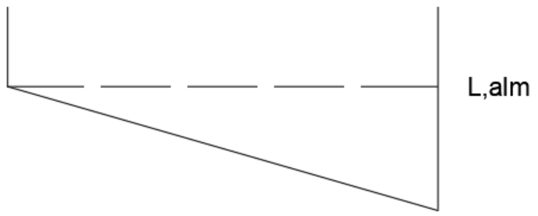


Figure 35. Storage length on cross-section

The first methodology is when the length of the water sheet for that slope is less than the SUDS storage length. In this way, the water is stored with a triangle-shaped section and the volume of water capable of managing the SUDS would be determined as:

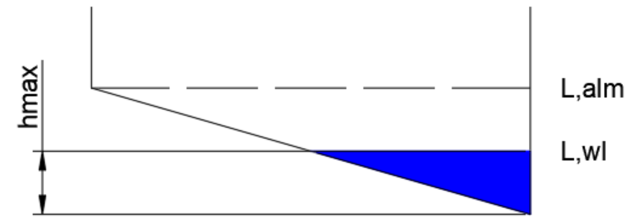


Figure 36. Storage volume if Lwl<Lalm

$$V_{SUDS} = \frac{1}{2} * L_{wl} * h_{max} * b * n * N$$

In the case where the length of the water sheet for the slope is greater than the storage length, then the volume of water is stored following the section of a trapezoid, which would be calculated as follows:

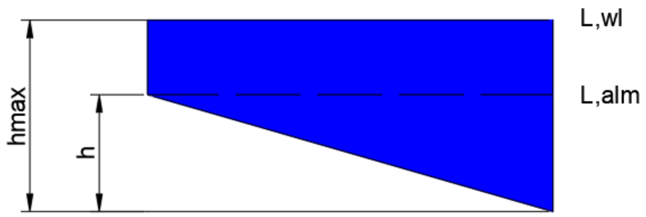


Figure 37. Storage volume if Lwl>Lalm

$$V_{SUDS} = [(h_{max} - h) + \frac{1}{2} * h] * L_{alm} * b * n * N$$

4.4.5.3 Parameters for determining the storage volumen

For each cross-section type, the void ratio has been determined. In the case of permeable pavements, the storage layer is composed of a single material, gravels, which have been determined to have a void ratio of 0.5 (Sañudo-Fontaneda et al., 2014).

$$n_{pp} = 0{,}5$$

$$e_{pp} = 450\ mm$$

Although the rain garden allows for surface storage in cases of extreme rainfall events, we will consider for the verification that it is only the storage layer that stores the water, also composed of gravels, which gives us:

$$n_{jll} = 0{,}5$$

$$e_{jll} = 450\ mm$$

In the case of the infiltration basins, the equivalent void ratio has been determined. This is due to the fact that there are layers with different materials that have different void indices, and for the calculation, the sum of all the layers is unified into one layer, and the equivalent void index of all the layers is unified into one thickness layer

$$n_{equivalente} = \frac{\sum_{i=1}^{n} e_i * n_i}{\sum_{i=1}^{n} e_i}$$

$$n_{ab} = \frac{1 * 600 + 0{,}5 * 300 + 0{,}3 * 450}{1350} = 0{,}66$$

$$e_{ab} = 1350\ mm$$

4.4.5.4 Result of volumetric efficiency analysis

Table 24. SuDS volumetric verification. Catchment N1

SUDS CATCHMENT N1																				
Type	Code	A,SUDS	L	b	N	U	n	hmax	h	slope	Lalm	Lwl	Lalm<Lwl	V, SUDS	Subcatch	A,imp	V80	V90	V80	V90
		m2	m	m	-	-	-	m	m	-	m	m	-	m3	-	m2	m3	m3		
PERVIOUS PAVEMENT	**PP101**	143,40	23,9	3	1	2	0,5	0,45	0,13	0,0054	47,8	83,3	SI	55,3	C101	1300,28	19,50	32,51	YES	YES
	PP102	480,30	80,1	3	1	2	0,5	0,45	0,43	0,0054	160,1	83,3	NO	56,3	C102	2281,05	34,22	57,03	YES	NO
	PP103	463,20	77,2	3	1	2	0,5	0,45	0,42	0,0054	154,4	83,3	NO	56,3	C103	2155,18	32,33	53,88	YES	YES
	PP104	472,20	78,7	3	1	2	0,5	0,45	0,42	0,0054	157,4	83,3	NO	56,3	C104	2666,12	39,99	66,65	YES	NO
	PP105	577,20	96,2	3	1	2	0,5	0,45	0,52	0,0054	192,4	83,3	NO	56,3	C105	2600,73	39,01	65,02	YES	NO
	PP106	489,60	81,6	3	1	2	0,5	0,45	0,44	0,0054	163,2	83,3	NO	56,3	C106	2204,25	33,06	55,11	YES	YES
	PP107	477,90	79,7	3	1	2	0,5	0,45	0,43	0,0054	159,3	83,3	NO	56,3	C107	2502,34	37,54	62,56	YES	NO
	PP116	146,70	48,9	3	1	1	0,5	0,45	0,30	0,0061	48,9	73,8	SI	22,1	C116	994,93	14,92	24,87	YES	NO
	PP117	138,60	46,2	3	1	1	0,5	0,45	0,36	0,0078	46,2	57,7	SI	18,7	C117	978,62	14,68	24,47	YES	NO
	PP118	213,60	71,2	3	1	1	0,5	0,45	0,56	0,0078	71,2	57,7	NO	19,5	C118	1223,88	18,36	30,60	YES	NO
	PP119	171,30	57,1	3	1	1	0,5	0,45	0,45	0,0078	57,1	57,7	SI	19,5	C119	1061,17	15,92	26,53	YES	NO
	PP120	204,00	68,0	3	1	1	0,5	0,45	0,22	0,0032	68	140,6	SI	34,8	C120	1577,68	23,67	39,44	YES	NO
	PP121	180,30	60,1	3	1	1	0,5	0,45	0,19	0,0032	60,1	140,6	SI	31,9	C121	1391,42	20,87	34,79	YES	NO
	PP122	210,60	70,2	3	1	1	0,5	0,45	0,22	0,0032	70,2	140,6	SI	35,6	C122	1598,83	23,98	39,97	YES	NO
	PP123	259,20	43,2	3	1	2	0,5	0,45	0,65	0,0151	86,4	29,8	NO	20,1	C123	1789,89	26,85	44,75	NO	NO
	PP124	222,66	54,1	3	1	2	0,5	0,45	0,82	0,0151	108,2	29,8	NO	20,1	C124	1703,45	25,55	42,59	NO	NO
RAIN GARDENS	**JLL108**	593,40	43,0	13,7	1	1	0,5	0,45	0,18	0,0042	43	107,1	SI	105,9	C108	738,33	11,07	18,46	YES	YES
	JLL109	1017,50	78,3	13,1	1	1	0,5	0,45	0,33	0,0042	78,3	107,1	SI	146,5	C109	1382,08	20,73	34,55	YES	YES
	JLL110	835,20	69,6	12	1	1	0,5	0,45	0,29	0,0042	69,6	107,1	SI	126,9	C110	1275,90	19,14	31,90	YES	YES
	JLL111	755,20	72,0	11	1	1	0,5	0,45	0,30	0,0042	72	107,1	SI	118,3	C111	1400,29	21,00	35,01	YES	YES
	JLL112	363,10	62,4	5,5	1	1	0,5	0,45	0,07	0,0012	62,4	375,0	SI	70,8	C112	1170,75	17,56	29,27	YES	YES
	JLL113	208,50	71,2	3	1	1	0,5	0,45	0,09	0,0012	71,2	375,0	SI	43,5	C113	671,50	10,07	16,79	YES	YES
	JLL114	156,30	39,4	4	1	1	0,5	0,45	0,05	0,0012	39,4	375,0	SI	33,6	C114	432,02	6,48	10,80	YES	YES
	JLL115	172,50	36,3	4,8	1	1	0,5	0,45	0,04	0,0012	36,3	375,0	SI	37,3	C115	425,01	6,38	10,63	YES	YES
INF	**IN01**	12402,34	157,5	78,75	1	1	0,57	1,35	0,78745	0,005	157,49	270,0	SI	6760,2	N1	13423,78	1737,71	2896,19	YES	YES

Table 25. SuDS volumetric verification. Catchment N2

SUDS CATCHMENT N2																				
Type	Code	A,SUDS	L	b	N	U	n	hmax	h	slope	Lalm	Lwl	Lalm<Lwl	V, SUDS	Subcatch	A,imp	V80	V90	V80	V90
		m2	m	m	-	-	-	m	m	-	m	m	-	m3	-	m2	m3	m3		
PERVIOUS PAVEMENTS	PP201	394,20	65,7	3	1	2	0,5	0,45	0,14	0,0021	131,4	214,3	SI	150,2	C201	2845,03	42,68	71,13	YES	YES
	PP202	575,40	95,9	3	1	2	0,5	0,45	0,20	0,0021	191,8	214,3	SI	201,0	C202	3652,54	54,79	91,31	YES	YES
	PP203	581,40	96,9	3	1	2	0,5	0,45	0,20	0,0021	193,8	214,3	SI	202,5	C203	3988,86	59,83	99,72	YES	YES
	PP204	546,30	91,1	3	1	2	0,5	0,45	0,19	0,0021	182,1	214,3	SI	193,6	C204	3574,60	53,62	89,37	YES	YES
	PP205	485,10	80,9	3	1	2	0,5	0,45	0,17	0,0021	161,7	214,3	SI	177,1	C205	3066,55	46,00	76,66	YES	YES
	PP206	527,10	87,9	3	1	2	0,5	0,45	0,18	0,0021	175,7	214,3	SI	188,6	C206	3751,73	56,28	93,79	YES	YES
	PP207	494,10	82,4	3	1	2	0,5	0,45	0,17	0,0021	164,7	214,3	SI	179,6	C207	3109,23	46,64	77,73	YES	YES
	PP208	388,20	64,7	3	1	2	0,5	0,45	0,14	0,0021	129,4	214,3	SI	148,3	C208	1621,12	24,32	40,53	YES	YES
	PP209	390,60	65,1	3	1	2	0,5	0,45	0,14	0,0021	130,2	214,3	SI	149,1	C209	2700,58	40,51	67,51	YES	YES
	PP219	333,60	55,6	3	1	2	0,5	0,45	0,62	0,0112	111,2	40,2	NO	27,1	C219	1238,72	18,58	30,97	YES	NO
	PP220	411,30	68,6	3	1	2	0,5	0,45	0,77	0,0112	137,1	40,2	NO	27,1	C220	1492,70	22,39	37,32	YES	NO
	PP221	403,80	67,3	3	1	2	0,5	0,45	0,75	0,0112	134,6	40,2	NO	27,1	C221	1446,70	21,70	36,17	YES	NO
	PP222	568,80	94,8	3	1	2	0,5	0,45	0,82	0,0086	189,6	52,3	NO	35,3	C222	2339,30	35,09	58,48	YES	NO
	PP223	313,80	52,3	3	1	2	0,5	0,45	0,45	0,0086	104,6	52,3	NO	35,3	C223	1315,87	19,74	32,90	YES	YES
	PP224	222,60	37,1	3	1	2	0,5	0,45	0,32	0,0086	74,2	52,3	NO	35,3	C224	943,75	14,16	23,59	YES	YES
	PP225	409,20	68,2	3	1	2	0,5	0,45	0,59	0,0086	136,4	52,3	NO	35,3	C225	2062,85	30,94	51,57	YES	NO
	PP226	292,50	48,8	3	1	2	0,5	0,45	0,55	0,0112	97,5	40,2	NO	27,1	C226	1040,28	15,60	26,01	YES	YES
	PP227	438,30	73,1	3	1	2	0,5	0,45	1,01	0,0138	146,1	32,6	NO	22,0	C227	2843,31	42,65	71,08	NO	NO
	PP228	436,50	72,8	3	1	2	0,5	0,45	1,00	0,0138	145,5	32,6	NO	22,0	C228	2516,11	37,74	62,90	NO	NO
RAIN GARDENS	JLL210	99,50	48,4	2,05	1	1	0,5	0,45	0,33	0,0068	48,44	66,2	SI	14,2	C210	858,82	12,88	21,47	YES	NO
	JLL211	274,80	78,1	3,52	1	1	0,5	0,45	0,53	0,0068	78,11	66,2	NO	26,2	C211	1215,79	18,24	30,39	YES	NO
	JLL212	516,30	89,5	5,77	1	1	0,5	0,45	0,61	0,0068	89,49	66,2	NO	43,0	C212	1332,32	19,98	33,31	YES	YES
	JLL213	631,50	79,8	7,91	1	1	0,5	0,45	0,54	0,0068	79,82	66,2	NO	58,9	C213	1117,98	16,77	27,95	YES	YES
	JLL214	853,20	87,7	9,73	1	1	0,5	0,45	0,60	0,0068	87,67	66,2	NO	72,5	C214	1179,36	17,69	29,48	YES	YES
	JLL215	1049,90	93,5	11,23	1	1	0,5	0,45	0,64	0,0068	93,46	66,2	NO	83,6	C215	1229,16	18,44	30,73	YES	YES
	JLL216	871,10	90,5	9,62	1	1	0,5	0,45	0,62	0,0068	90,54	66,2	NO	71,6	C216	1394,97	20,92	34,87	YES	YES
	JLL217	750,20	83,8	8,95	1	1	0,5	0,45	0,57	0,0068	83,83	66,2	NO	66,6	C217	1749,53	26,24	43,74	YES	YES
	JLL218	726,30	85,8	8,47	1	1	0,5	0,45	0,58	0,0068	85,79	66,2	NO	63,0	C218	2008,36	30,13	50,21	YES	YES
INF	IN02	23003,05	139,4	127,25	1	1	0,66	1,35	0,697	0,005	139,4	270,0	SI	11725,1	N2	247356,30	3710,34	6183,91	YES	YES

Table 26. SuDS volumetric verification on pervious pavements. Catchment N3

SUDS CATCHMENT N3																				
Type	Code	A,SUDS	L	b	N	U	n	hmax	h	slope	Lalm	Lwl	Lalm<Lwl	V, SUDS	Subcatch	A,imp	V80	V90	V80	V90
		m2	m	m	-	-	-	m	m	-	m	m	-	m3	-	m2	m3	m3		
PERVIOUS PAVEMENTS	PP301	189,30	63,1	3,00	1	1	0,5	0,45	0,07	0,0011	63,1	409,1	SI	39,3	C301	1220,07	18,30	30,50	YES	YES
	PP302	132,60	44,2	3,00	1	1	0,5	0,45	0,05	0,0011	44,2	409,1	SI	28,2	C302	866,31	12,99	21,66	YES	YES
	PP303	466,80	77,8	3,00	1	2	0,5	0,45	0,51	0,0066	155,6	68,2	NO	46,0	C303	3830,71	57,46	95,77	NO	NO
	PP304	582,30	97,1	3,00	1	2	0,5	0,45	0,64	0,0066	194,1	68,2	NO	46,0	C304	4898,39	73,48	122,46	NO	NO
	PP305	468,00	78,0	3,00	1	2	0,5	0,45	0,51	0,0066	156	68,2	NO	46,0	C305	4518,68	67,78	112,97	NO	NO
	PP306	309,00	51,5	3,00	1	2	0,5	0,45	0,48	0,0093	103	48,4	NO	32,7	C306	2648,99	39,73	66,22	NO	NO
	PP307	309,30	51,6	3,00	1	2	0,5	0,45	0,48	0,0093	103,1	48,4	NO	32,7	C307	2749,60	41,24	68,74	NO	NO
	PP308	567,60	94,6	3,00	1	2	0,5	0,45	0,64	0,0068	189,2	66,2	NO	44,7	C308	4696,07	70,44	117,40	NO	NO
	PP309	487,20	81,2	3,00	1	2	0,5	0,45	0,65	0,008	162,4	56,3	NO	38,0	C309	3955,52	59,33	98,89	NO	NO
	PP310	60,90	20,3	3,00	1	1	0,5	0,45	0,14	0,0069	20,3	65,2	SI	11,6	C310	482,48	7,24	12,06	YES	NO
	PP311	208,50	69,5	3,00	1	1	0,5	0,45	0,34	0,0049	69,5	91,8	SI	29,2	C311	1637,71	24,57	40,94	YES	NO
	PP312	272,70	90,9	3,00	1	1	0,5	0,45	0,45	0,0049	90,9	91,8	SI	31,0	C312	2060,55	30,91	51,51	YES	NO
	PP313	223,20	74,4	3,00	1	1	0,5	0,45	0,36	0,0049	74,4	91,8	SI	29,9	C313	1557,61	23,36	38,94	YES	NO
	PP314	264,30	88,1	3,00	1	1	0,5	0,45	0,43	0,0049	88,1	91,8	SI	30,9	C314	1966,46	29,50	49,16	YES	NO
	PP315	480,30	80,1	3,00	1	2	0,5	0,45	0,55	0,0069	160,1	65,2	NO	44,0	C315	1786,16	26,79	44,65	YES	NO
	PP316	524,10	87,4	3,00	1	2	0,5	0,45	0,60	0,0069	174,7	65,2	NO	44,0	C316	2190,85	32,86	54,77	YES	NO
	PP317	316,80	52,8	3,00	1	2	0,5	0,45	0,36	0,0069	105,6	65,2	NO	44,0	C317	1722,98	25,84	43,07	YES	YES
	PP318	216,60	36,1	3,00	1	2	0,5	0,45	0,25	0,0069	72,2	65,2	NO	44,0	C318	1216,97	18,25	30,42	YES	YES
	PP320	178,20	59,4	3,00	1	1	0,5	0,45	0,05	0,0009	59,4	500,0	SI	37,7	C320	1229,94	18,45	30,75	YES	YES
	PP321	176,70	58,9	3,00	1	1	0,5	0,45	0,05	0,0009	58,9	500,0	SI	37,4	C321	1335,45	20,03	33,39	YES	YES
	PP322	154,20	51,4	3,00	1	1	0,5	0,45	0,05	0,0009	51,4	500,0	SI	32,9	C322	1220,05	18,30	30,50	YES	YES
	PP323	153,90	51,3	3,00	1	1	0,5	0,45	0,05	0,0009	51,3	500,0	SI	32,9	C323	1195,09	17,93	29,88	YES	YES
	PP324	84,90	28,3	3,00	1	1	0,5	0,45	0,03	0,0009	28,3	500,0	SI	18,6	C324	899,24	13,49	22,48	YES	NO
	PP325	555,90	92,7	3,00	1	2	0,5	0,45	0,08	0,0009	185,3	500,0	SI	227,0	C325	1682,75	25,24	42,07	YES	YES
	PP326	421,80	70,3	3,00	1	2	0,5	0,45	0,06	0,0009	140,6	500,0	SI	176,5	C326	1278,70	19,18	31,97	YES	YES
	PP327	480,00	80,0	3,00	1	2	0,5	0,45	0,07	0,0009	160	500,0	SI	198,7	C327	1365,90	20,49	34,15	YES	YES
	PP328	113,40	37,8	3,00	1	1	0,5	0,45	0,12	0,0031	37,8	145,2	SI	22,2	C328	700,55	10,51	17,51	YES	YES
	PP329	237,90	79,3	3,00	1	1	0,5	0,45	0,25	0,0031	79,3	145,2	SI	38,9	C329	1044,24	15,66	26,11	YES	YES
	PP330	233,10	77,7	3,00	1	1	0,5	0,45	0,24	0,0031	77,7	145,2	SI	38,4	C330	1022,86	15,34	25,57	YES	YES
	PP331	498,00	83,0	3,00	1	2	0,5	0,45	0,26	0,0031	166	145,2	NO	98,0	C331	1082,86	16,24	27,07	YES	YES
	PP332	458,40	76,4	3,00	1	2	0,5	0,45	0,24	0,0031	152,8	145,2	NO	98,0	C332	1241,05	18,62	31,03	YES	YES
	PP333	735,10	25,1	29,33	1	2	0,5	0,45	0,08	0,0031	50,12	145,2	SI	604,5	C333	1013,70	15,21	25,34	YES	YES
	PP334	2133,70	69,8	30,55	1	2	0,5	0,45	0,22	0,0031	139,68	145,2	SI	1458,4	C334	3228,01	48,42	80,70	YES	YES
	PP335	66,60	22,2	3,00	1	1	0,5	0,45	0,03	0,0014	22,2	321,4	SI	14,5	C335	261,14	3,92	6,53	YES	YES
	PP336	242,70	80,9	3,00	1	1	0,5	0,45	0,11	0,0014	80,9	321,4	SI	47,7	C336	917,85	13,77	22,95	YES	YES
	PP337	236,10	78,7	3,00	1	1	0,5	0,45	0,11	0,0014	78,7	321,4	SI	46,6	C337	1135,13	17,03	28,38	YES	YES
	PP338	286,80	95,6	3,00	1	1	0,5	0,45	0,13	0,0014	95,6	321,4	SI	54,9	C338	1269,99	19,05	31,75	YES	YES
	PP339	253,20	84,4	3,00	1	1	0,5	0,45	0,12	0,0014	84,4	321,4	SI	49,5	C339	1273,16	19,10	31,83	YES	YES
	PP340	220,80	73,6	3,00	1	1	0,5	0,45	0,10	0,0014	73,6	321,4	SI	44,0	C340	1214,10	18,21	30,35	YES	YES
	PP341	185,40	61,8	3,00	1	1	0,5	0,45	0,09	0,0014	61,8	321,4	SI	37,7	C341	1097,36	16,46	27,43	YES	YES
	PP342	80,10	26,7	3,00	1	1	0,5	0,45	0,05	0,002	26,7	225,0	SI	17,0	C342	353,33	5,30	8,83	YES	YES
	PP343	232,50	77,5	3,00	1	1	0,5	0,45	0,16	0,002	77,5	225,0	SI	43,3	C343	1358,71	20,38	33,97	YES	YES
	PP344	305,70	101,9	3,00	1	1	0,5	0,45	0,20	0,002	101,9	225,0	SI	53,2	C344	826,55	12,40	20,66	YES	YES
	PP345	294,60	98,2	3,00	1	1	0,5	0,45	0,20	0,002	98,2	225,0	SI	51,8	C345	808,52	12,13	20,21	YES	YES
	PP346	253,80	84,6	3,00	1	1	0,5	0,45	0,17	0,002	84,6	225,0	SI	46,4	C346	697,60	10,46	17,44	YES	YES
	PP347	89,10	29,7	3,00	1	1	0,5	0,45	0,23	0,0079	29,7	57,0	SI	14,8	C347	508,12	7,62	12,70	YES	YES
	PP348	207,90	69,3	3,00	1	1	0,5	0,45	0,55	0,0079	69,3	57,0	NO	19,2	C348	1220,67	18,31	30,52	YES	NO
	PP349	203,40	67,8	3,00	1	1	0,5	0,45	0,49	0,0072	67,8	62,5	NO	21,1	C349	1379,15	20,69	34,48	YES	NO
	PP350	207,30	69,1	3,00	1	1	0,5	0,45	0,40	0,0058	69,1	77,6	SI	25,9	C350	1022,72	15,34	25,57	YES	YES
	PP351	177,00	59,0	3,00	1	1	0,5	0,45	0,34	0,0058	59	77,6	SI	24,7	C351	845,05	12,68	21,13	YES	YES
	PP352	189,60	63,2	3,00	1	1	0,5	0,45	0,37	0,0058	63,2	77,6	SI	25,3	C352	861,85	12,93	21,55	YES	YES
	PP353	162,90	54,3	3,00	1	1	0,5	0,45	0,31	0,0058	54,3	77,6	SI	23,8	C353	666,93	10,00	16,67	YES	YES
	PP354	47,10	15,7	3,00	1	1	0,5	0,45	0,00	0,0001	15,7	4500,0	SI	10,6	C354	334,32	5,01	8,36	YES	YES
	PP355	193,80	64,6	3,00	1	1	0,5	0,45	0,01	0,0001	64,6	4500,0	SI	43,3	C355	1178,31	17,67	29,46	YES	YES
	PP356	243,30	81,1	3,00	1	1	0,5	0,45	0,01	0,0001	81,1	4500,0	SI	54,2	C356	1285,97	19,29	32,15	YES	YES
	PP357	268,20	89,4	3,00	1	1	0,5	0,45	0,01	0,0001	89,4	4500,0	SI	59,7	C357	975,67	14,64	24,39	YES	YES
	PP358	154,20	51,4	3,00	1	1	0,5	0,45	0,01	0,0001	51,4	4500,0	SI	34,5	C358	455,55	6,83	11,39	YES	YES
	PP359	302,40	50,4	3,00	1	2	0,5	0,45	0,01	0,0001	100,8	4500,0	SI	135,3	C359	2583,57	38,75	64,59	YES	YES
	PP360	599,40	99,9	3,00	1	2	0,5	0,45	0,01	0,0001	199,8	4500,0	SI	266,7	C360	3849,28	57,74	96,23	YES	YES
	PP361	559,20	93,2	3,00	1	2	0,5	0,45	0,01	0,0001	186,4	4500,0	SI	249,0	C361	4182,60	62,74	104,56	YES	YES
	PP362	452,70	75,5	3,00	1	2	0,5	0,45	0,01	0,0001	150,9	4500,0	SI	202,0	C362	5296,36	79,45	132,41	YES	YES
	PP363	166,50	27,8	3,00	1	2	0,5	0,45	0,26	0,0094	55,5	47,9	NO	32,3	C363	530,85	7,96	13,27	YES	YES
	PP364	415,50	69,3	3,00	1	2	0,5	0,45	0,65	0,0094	138,5	47,9	NO	32,3	C364	1313,72	19,71	32,84	YES	NO
	PP365	426,90	71,2	3,00	1	2	0,5	0,45	0,67	0,0094	142,3	47,9	NO	32,3	C365	1331,39	19,97	33,28	YES	NO
	PP366	544,50	90,8	3,00	1	2	0,5	0,45	0,81	0,0089	181,5	50,6	NO	34,1	C366	1667,45	25,01	41,69	YES	NO
	PP367	570,00	95,0	3,00	1	2	0,5	0,45	0,79	0,0083	190	54,2	NO	36,6	C367	2080,79	31,21	52,02	YES	NO
	PP368	467,40	77,9	3,00	1	2	0,5	0,45	0,73	0,0094	155,8	47,9	NO	32,3	C368	2069,69	31,05	51,74	YES	NO
	PP369	52,20	17,4	3,00	1	1	0,5	0,45	0,18	0,0106	17,4	42,5	SI	9,3	C369	164,70	2,47	4,12	YES	YES
	PP370	206,10	68,7	3,00	1	1	0,5	0,45	0,73	0,0106	68,7	42,5	NO	14,3	C370	560,28	8,40	14,01	YES	YES
	PP371	173,10	57,7	3,00	1	1	0,5	0,45	0,61	0,0106	57,7	42,5	NO	14,3	C371	823,60	12,35	20,59	YES	NO
	PP372	125,70	21,0	3,00	1	2	0,5	0,45	0,27	0,0131	41,9	34,4	NO	23,2	C372	724,40	10,87	18,11	YES	YES
	PP373	241,50	40,3	3,00	1	2	0,5	0,45	0,53	0,0131	80,5	34,4	NO	23,2	C373	927,53	13,91	23,19	YES	NO
	PP374	271,80	45,3	3,00	1	2	0,5	0,45	0,59	0,0131	90,6	34,4	NO	23,2	C374	1327,03	19,91	33,18	YES	NO
	PP375	263,40	87,8	3,00	1	1	0,5	0,45	0,54	0,0061	87,8	73,8	NO	24,9	C375	1437,72	21,57	35,94	YES	NO
	PP376	275,70	91,9	3,00	1	1	0,5	0,45	0,56	0,0061	91,9	73,8	NO	24,9	C376	1532,68	22,99	38,32	YES	NO
	PP377	258,60	86,2	3,00	1	1	0,5	0,45	0,53	0,0061	86,2	73,8	NO	24,9	C377	1403,01	21,05	35,08	YES	NO
	PP378	217,50	72,5	3,00	1	1	0,5	0,45	0,44	0,0061	72,5	73,8	SI	24,9	C378	1201,72	18,03	30,04	YES	NO
	PP379	129,30	21,6	3,00	1	2	0,5	0,45	0,02	0,0008	43,1	562,5	SI	57,1	C379	607,06	9,11	15,18	YES	YES
	PP380	495,30	82,6	3,00	1	2	0,5	0,45	0,07	0,0008	165,1	562,5	SI	206,5	C380	2370,36	35,56	59,26	YES	YES
	PP381	381,60	63,6	3,00	1	2	0,5	0,45	0,05	0,0008	127,2	562,5	SI	162,0	C381	2595,10	38,93	64,88	YES	YES
	PP382	519,30	86,6	3,00	1	2	0,5	0,45	0,26	0,003	173,1	150,0	NO	101,3	C382	2481,29	37,22	62,03	YES	YES
	PP383	588,30	98,1	3,00	1	2	0,5	0,45	0,29	0,003	196,1	150,0	NO	101,3	C383	2491,98	37,38	62,30	YES	YES
	PP384	561,90	93,7	3,00	1	2	0,5	0,45	0,28	0,003	187,3	150,0	NO	101,3	C384	2266,74	34,00	56,67	YES	YES
	PP385	363,60	60,6	3,00	1	2	0,5	0,45	0,18	0,003	121,2	150,0	SI	130,6	C385	2424,81	36,37	60,62	YES	YES
	PP386	411,60	68,6	3,00	1	2	0,5	0,45	0,25	0,0036	137,2	125,0	NO	84,4	C386	1690,05	25,35	42,25	YES	YES
	PP387	480,60	80,1	3,00	1	2	0,5	0,45	0,29	0,0036	160,2	125,0	NO	84,4	C387	1677,75	25,17	41,94	YES	YES
	PP388	285,00	47,5	3,00	1	2	0,5	0,45	0,17	0,0036	95	125,0	SI	103,9	C388	1253,03	18,80	31,33	YES	YES
	PP389	175,80	58,6	3,00	1	1	0,5	0,45	0,18	0,003	58,6	150,0	SI	31,8	C389	1135,57	17,03	28,39	YES	YES
	PP390	218,10	72,7	3,00	1	1	0,5	0,45	0,22	0,003	72,7	150,0	SI	37,2	C390	1478,30	22,17	36,96	YES	YES
	PP391	174,60	58,2	3,00	1	1	0,5	0,45	0,17	0,003	58,2	150,0	SI	31,7	C391	1252,63	18,79	31,32	YES	YES
	PP392	263,40	87,8	3,00	1	1	0,5	0,45	0,44	0,005	87,8	90,0	SI	30,4	C392	1983,11	29,75	49,58	YES	NO
	PP393	208,20	69,4	3,00	1	1	0,5	0,45	0,35	0,005	69,4	90,0	SI	28,8	C393	2006,79	30,10	50,17	NO	NO
	PP394	501,30	83,6	3,00	1	2	0,5	0,45	0,12	0,0014	167,1	321,4	SI	196,3	C394	4379,44	65,69	109,49	YES	YES
	PP395	474,00	79,0	3,00	1	2	0,5	0,45	0,11	0,0014	158	321,4	SI	187,1	C395	4534,77	68,02	113,37	YES	YES
	PP396	218,40	72,8	3,00	1	1	0,5	0,45	0,31	0,0042	72,8	107,1	SI	32,4	C396	1310,55	19,66	32,76	YES	NO
	PP397	209,10	69,7	3,00	1	1	0,5	0,45	0,29	0,0042	69,7	107,1	SI	31,7	C397	1469,63	22,04	36,74	YES	NO
	PP398	236,40	78,8	3,00	1	1	0,5	0,45	0,09	0,0011	78,8	409,1	SI	48,1	C398	1332,27	19,98	33,31	YES	YES

Table 27. SuDS volumetric verification on rain gardens. Catchment N3

SUDS CATCHMENT N3																			
Type	Code	A,SUDS	L	b	N	U	n	hmax	h	slope	Lalm	Lwl	Lalm<Lwl	V, SUDS	Subcatch	A,imp	V80	V90	V80	V90
		m2	m	m	-	-	-	m	m	-	m	m	-	m3	-	m2	m3	m3		
RAIN GARDENS	JLL303	233,46	77,82	3,00	1	1	0,5	0,75	0,61	0,0078	77,82	96,2	SI	52,1	C395	4534,77	68,02	113,37	NO	NO
	JLL304	291,09	97,03	3,00	1	1	0,5	0,75	0,76	0,0078	97,03	96,2	NO	54,1	C303	3830,71	57,46	95,77	NO	NO
	JLL305	212,13	70,71	3,00	1	1	0,5	0,75	0,55	0,0078	70,71	96,2	SI	50,3	C304	4898,39	73,48	122,46	NO	NO
	JLL306	147,00	49	3,00	1	1	0,5	0,75	0,46	0,0093	49	80,6	SI	38,4	C305	4518,68	67,78	112,97	NO	NO
	JLL307	142,20	47,4	3,00	1	1	0,5	0,75	0,44	0,0093	47,4	80,6	SI	37,7	C306	2648,99	39,73	66,22	NO	NO
	JLL308	262,05	87,35	3,00	1	1	0,5	0,75	0,83	0,0095	87,35	78,9	NO	44,4	C307	2749,60	41,24	68,74	YES	NO
	JLL309	196,26	65,42	3,00	1	1	0,5	0,75	1,03	0,0157	65,42	47,8	NO	26,9	C308	2696,07	40,44	67,40	NO	NO
	JLL310	165,60	20,45	8,10	1	1	0,5	0,75	0,14	0,0069	20,45	108,7	SI	56,3	C310	482,48	7,24	12,06	YES	YES
	JLL311	533,50	70,31	7,59	1	1	0,5	0,75	0,49	0,0069	70,31	108,7	SI	135,4	C310	482,48	7,24	12,06	YES	YES
	JLL312	629,10	90,02	6,99	1	1	0,5	0,75	0,62	0,0069	90,02	108,7	SI	138,2	C311	1637,71	24,57	40,94	YES	YES
	JLL313	549,80	74,55	7,37	1	1	0,5	0,75	0,51	0,0069	74,55	108,7	SI	135,5	C312	2060,55	30,91	51,51	YES	YES
	JLL314	421,50	96,33	4,38	1	1	0,5	0,75	0,66	0,0069	96,33	108,7	SI	88,0	C313	1557,61	23,36	38,94	YES	YES
	JLL315	1070,20	80,4	13,31	1	1	0,5	0,75	0,55	0,0069	80,4	108,7	SI	252,9	C315	1786,16	26,79	44,65	YES	YES
	JLL316	1050,00	77,39	13,57	1	1	0,5	0,75	0,53	0,0069	77,39	108,7	SI	253,6	C315	1786,16	26,79	44,65	YES	YES
	JLL317	158,67	52,89	3,00	1	1	0,5	0,75	0,36	0,0069	52,89	108,7	SI	45,0	C316	2190,85	32,86	54,77	YES	NO
	JLL318	107,10	35,7	3,00	1	1	0,5	0,75	0,25	0,0069	35,7	108,7	SI	33,6	C317	1722,98	25,84	43,07	YES	NO
	JLL319	70,50	24,69	2,86	1	1	0,5	0,75	0,02	0,0009	24,69	833,3	SI	26,0	C314	1966,46	29,50	49,16	NO	NO
	JLL320	76,20	57,88	1,32	1	1	0,5	0,75	0,05	0,0009	57,88	833,3	SI	27,6	C319	513,92	7,71	12,85	YES	YES
	JLL321	148,00	60,89	2,43	1	1	0,5	0,75	0,05	0,0009	60,89	833,3	SI	53,5	C320	1229,94	18,45	30,75	YES	YES
	JLL322	273,40	53,72	5,09	1	1	0,5	0,75	0,05	0,0009	53,72	833,3	SI	99,2	C321	1335,45	20,03	33,39	YES	YES
	JLL323	359,70	56,99	6,31	1	1	0,5	0,75	0,05	0,0009	56,99	833,3	SI	130,3	C322	1220,05	18,30	30,50	YES	YES
	JLL324	238,80	44,51	5,37	1	1	0,5	0,75	0,04	0,0009	44,51	833,3	SI	87,2	C323	1195,09	17,93	29,88	YES	YES
	JLL328	156,30	43,22	3,62	1	1	0,5	0,75	0,13	0,0031	43,22	241,9	SI	53,4	C324	899,24	13,49	22,48	YES	YES
	JLL329	608,00	80,26	7,58	1	1	0,5	0,75	0,25	0,0031	80,26	241,9	SI	190,2	C328	700,55	10,51	17,51	YES	YES
	JLL330	628,70	88,36	7,12	1	1	0,5	0,75	0,27	0,0031	88,36	241,9	SI	192,7	C329	1044,24	15,66	26,11	YES	YES
	JLL335	172,30	23,86	7,22	1	1	0,5	0,75	0,03	0,0014	23,86	535,7	SI	63,2	C330	1022,86	15,34	25,57	YES	YES
	JLL336	625,80	83,03	7,54	1	1	0,5	0,75	0,12	0,0014	83,03	535,7	SI	216,5	C335	261,14	3,92	6,53	YES	YES
	JLL337	1052,60	93,09	11,31	1	1	0,5	0,75	0,13	0,0014	93,09	535,7	SI	360,4	C336	917,85	13,77	22,95	YES	YES
	JLL342	336,30	28,99	11,60	1	1	0,5	0,45	0,06	0,002	28,99	225,0	SI	70,8	C337	1135,13	17,03	28,38	YES	YES
	JLL343	488,30	88,76	5,50	1	1	0,5	0,45	0,18	0,002	88,76	225,0	SI	88,2	C342	353,33	5,30	8,83	YES	YES
	JLL347	105,80	27,4	3,86	1	1	0,5	0,45	0,24	0,0087	27,4	51,7	SI	17,5	C343	1358,71	20,38	33,97	NO	NO
	JLL348	306,30	71,71	4,27	1	1	0,5	0,45	0,62	0,0087	71,71	51,7	NO	24,9	C347	508,12	7,62	12,70	YES	YES
	JLL349	447,00	82,75	5,40	1	1	0,5	0,45	0,72	0,0087	82,75	51,7	NO	31,4	C348	1220,67	18,31	30,52	YES	YES
	JLL354	102,00	18,17	5,61	1	1	0,5	0,45	0,00	0,0001	18,17	4500,0	SI	22,9	C349	1379,15	20,69	34,48	YES	NO
	JLL355	213,50	62,33	3,43	1	1	0,5	0,45	0,01	0,0001	62,33	4500,0	SI	47,7	C354	334,32	5,01	8,36	YES	YES
	JLL356	83,20	56,38	1,48	1	1	0,5	0,45	0,01	0,0001	56,38	4500,0	SI	18,6	C355	1178,31	17,67	29,46	YES	NO
	JLL359	128,28	42,76	3,00	1	1	0,5	0,45	0,00	0,0001	42,76	4500,0	SI	28,7	C356	1285,97	19,29	32,15	YES	NO
	JLL360	296,97	98,99	3,00	1	1	0,5	0,45	0,01	0,0001	98,99	4500,0	SI	66,1	C359	2583,57	38,75	64,59	YES	YES
	JLL361	281,94	93,98	3,00	1	1	0,5	0,45	0,01	0,0001	93,98	4500,0	SI	62,8	C360	3849,28	57,74	96,23	YES	NO
	JLL362	247,08	82,36	3,00	1	1	0,5	0,45	0,01	0,0001	82,36	4500,0	SI	55,1	C362	4182,60	62,74	104,56	NO	NO
	JLL368	203,13	67,71	3,00	1	1	0,5	0,45	0,64	0,0094	67,71	47,9	NO	16,2	C368	2069,69	31,05	51,74	NO	NO
	JLL379	64,98	21,66	3,00	1	1	0,5	0,45	0,02	0,0008	21,66	562,5	SI	14,3	C379	607,06	9,11	15,18	YES	NO
	JLL380	247,50	82,5	3,00	1	1	0,5	0,45	0,07	0,0008	82,5	562,5	SI	51,6	C379	607,06	9,11	15,18	YES	YES
	JLL381	190,14	63,38	3,00	1	1	0,5	0,45	0,05	0,0008	63,38	562,5	SI	40,4	C380	2370,36	35,56	59,26	YES	NO
	JLL382	245,04	81,68	3,00	1	1	0,5	0,45	0,25	0,003	81,68	150,0	SI	40,1	C382	2481,29	37,22	62,03	YES	NO
	JLL383	294,27	98,09	3,00	1	1	0,5	0,45	0,29	0,003	98,09	150,0	SI	44,6	C382	2481,29	37,22	62,03	YES	NO
	JLL384	280,74	93,58	3,00	1	1	0,5	0,45	0,28	0,003	93,58	150,0	SI	43,5	C383	2491,98	37,38	62,30	YES	NO
	JLL385	180,48	60,16	3,00	1	1	0,5	0,45	0,18	0,003	60,16	150,0	SI	32,5	C384	2266,74	34,00	56,67	NO	NO
	JLL386	206,49	68,83	3,00	1	1	0,5	0,45	0,25	0,0036	68,83	125,0	SI	33,7	C385	2424,81	36,37	60,62	NO	NO
	JLL387	240,09	80,03	3,00	1	1	0,5	0,45	0,29	0,0036	80,03	125,0	SI	36,7	C318/C385	3641,78	54,63	91,04	NO	NO
	JLL388	140,34	46,78	3,00	1	1	0,5	0,45	0,17	0,0036	46,78	125,0	SI	25,7	C387	1677,75	25,17	41,94	YES	NO
	JLL394	510,32	63,79	3,00	1	1	0,5	0,45	0,05	0,0008	63,79	562,5	SI	40,6	C394	4379,44	65,69	109,49	NO	NO
	JLL395	620,10	68,9	3,00	1	1	0,5	0,45	0,06	0,0008	68,9	562,5	SI	43,7	C394	4379,44	65,69	109,49	NO	NO

To solve the problem regarding the volumetric efficiency of the SuDS that have not met the quantity and quality criteria, the transversal compartments mentioned previously were added and the SuDS that did not meet the V_{80} and V_{90} criteria were divided into two sections.

After this modification it is obtained that all the SuDS of the project meet the quantity criterion and 91.9% meet the quality criterion, determining that the design is appropriate.

Table 28. SuDS modification with transversal compartments

MODIFICATION WITH TRANSVERSAL COMPARTIMENTS																			
Code	A,SUDS	L	b	N	U	n	hmax	h	slope	Lalm	Lwl	Lalm<Lwl?	V, SUDS	Subcatch	A,imp	V80	V90	V80	V90
	m2	m	m	-	-	-	m	m	-	m	m	-	m3	-	m2	m3	m3		
PP102	480,30	80,1	3	2	2	0,5	0,45	0,43227	0,0054	320,2	83,3	NO	112,5	C102	2281,05	34,22	57,03	YES	YES
PP104	472,20	78,7	3	2	2	0,5	0,45	0,42498	0,0054	314,8	83,3	NO	112,5	C104	2666,12	39,99	66,65	YES	YES
PP105	577,20	96,2	3	2	2	0,5	0,45	0,51948	0,0054	384,8	83,3	NO	112,5	C105	2600,73	39,01	65,02	YES	YES
PP107	477,90	79,7	3	2	2	0,5	0,45	0,43011	0,0054	318,6	83,3	NO	112,5	C107	2502,34	37,54	62,56	YES	YES
PP116	146,70	48,9	3	2	1	0,5	0,45	0,29829	0,0061	97,8	73,8	NO	49,8	C116	994,93	14,92	24,87	YES	YES
PP117	138,60	46,2	3	2	1	0,5	0,45	0,36036	0,0078	92,4	57,7	NO	38,9	C117	978,62	14,68	24,47	YES	YES
PP118	213,60	71,2	3	2	1	0,5	0,45	0,55536	0,0078	142,4	57,7	NO	38,9	C118	1223,88	18,36	30,60	YES	YES
PP119	171,30	57,1	3	2	1	0,5	0,45	0,44538	0,0078	114,2	57,7	NO	38,9	C119	1061,17	15,92	26,53	YES	YES
PP120	204,00	68,0	3	2	1	0,5	0,45	0,2176	0,0032	136	140,6	SI	139,2	C120	1577,68	23,67	39,44	YES	YES
PP121	180,30	60,1	3	2	1	0,5	0,45	0,19232	0,0032	120,2	140,6	SI	127,6	C121	1391,42	20,87	34,79	YES	YES
PP122	210,60	70,2	3	2	1	0,5	0,45	0,22464	0,0032	140,4	140,6	SI	142,2	C122	1598,83	23,98	39,97	YES	YES
PP123	259,20	43,2	3	2	2	0,5	0,45	0,65232	0,0151	172,8	29,8	NO	40,2	C123	1789,89	26,85	44,75	YES	NO
PP124	222,66	54,1	3	2	2	0,5	0,45	0,81691	0,0151	216,4	29,8	NO	40,2	C124	1703,45	25,55	42,59	YES	NO
PP219	333,60	55,6	3	2	2	0,5	0,45	0,62272	0,0112	222,4	40,2	NO	54,2	C219	1238,72	18,58	30,97	YES	YES
PP220	411,30	68,6	3	2	2	0,5	0,45	0,76776	0,0112	274,2	40,2	NO	54,2	C220	1492,70	22,39	37,32	YES	YES
PP221	403,80	67,3	3	2	2	0,5	0,45	0,75376	0,0112	269,2	40,2	NO	54,2	C221	1446,70	21,70	36,17	YES	YES
PP222	568,80	94,8	3	2	2	0,5	0,45	0,81528	0,0086	379,2	52,3	NO	70,6	C222	2339,30	35,09	58,48	YES	YES
PP225	409,20	68,2	3	2	2	0,5	0,45	0,58652	0,0086	272,8	52,3	NO	70,6	C225	2062,85	30,94	51,57	YES	YES
PP227	438,30	73,1	3	2	2	0,5	0,45	1,00809	0,0138	292,2	32,6	NO	44,0	C227	2843,31	42,65	71,08	YES	NO
PP228	436,50	72,8	3	2	2	0,5	0,45	1,00395	0,0138	291	32,6	NO	44,0	C228	2516,11	37,74	62,90	YES	NO
JLL210	99,50	48,4	2,05	2	1	0,5	0,45	0,329392	0,0068	96,88	66,2	NO	30,6	C210	858,82	12,88	21,47	YES	YES
JLL211	274,80	78,1	3,52	2	1	0,5	0,45	0,531148	0,0068	156,22	66,2	NO	52,4	C211	1215,79	18,24	30,39	YES	YES
PP303	466,80	77,8	3	2	2	0,5	0,45	0,51348	0,0066	155,6	68,2	NO	92,0	C303	3830,71	57,46	95,77	YES	NO
PP304	582,30	97,1	3	2	2	0,5	0,45	0,64053	0,0066	194,1	68,2	NO	92,0	C304	4898,39	73,48	122,46	YES	NO
PP305	468,00	78,0	3	2	2	0,5	0,45	0,5148	0,0066	156	68,2	NO	92,0	C305	4518,68	67,78	112,97	YES	NO
PP306	309,00	51,5	3	2	2	0,5	0,45	0,47895	0,0093	103	48,4	NO	65,3	C306	2648,99	39,73	66,22	YES	NO
PP307	309,30	51,6	3	2	2	0,5	0,45	0,479415	0,0093	103,1	48,4	NO	65,3	C307	2749,60	41,24	68,74	YES	NO
PP308	567,60	94,6	3	2	2	0,5	0,45	0,64328	0,0068	189,2	66,2	NO	89,3	C308	4696,07	70,44	117,40	YES	NO
PP309	487,20	81,2	3	2	2	0,5	0,45	0,6496	0,008	162,4	56,3	NO	75,9	C309	3955,52	59,33	98,89	YES	NO
PP310	60,90	20,3	3	2	1	0,5	0,45	0,14007	0,0069	20,3	65,2	SI	23,1	C310	482,48	7,24	12,06	YES	YES
PP311	208,50	69,5	3	2	1	0,5	0,45	0,34055	0,0049	69,5	91,8	SI	58,3	C311	1637,71	24,57	40,94	YES	YES
PP312	272,70	90,9	3	2	1	0,5	0,45	0,44541	0,0049	90,9	91,8	SI	62,0	C312	2060,55	30,91	51,51	YES	YES
PP313	223,20	74,4	3	2	1	0,5	0,45	0,36456	0,0049	74,4	91,8	SI	59,8	C313	1557,61	23,36	38,94	YES	YES
PP314	264,30	88,1	3	2	1	0,5	0,45	0,43169	0,0049	88,1	91,8	SI	61,9	C314	1966,46	29,50	49,16	YES	YES
PP315	480,30	80,1	3	2	2	0,5	0,45	0,552345	0,0069	160,1	65,2	NO	88,0	C315	1786,16	26,79	44,65	YES	YES
PP316	524,10	87,4	3	2	2	0,5	0,45	0,602715	0,0069	174,7	65,2	NO	88,0	C316	2190,85	32,86	54,77	YES	YES
PP324	84,90	28,3	3	2	1	0,5	0,45	0,02547	0,0009	28,3	500,0	SI	37,1	C324	899,24	13,49	22,48	YES	YES
PP348	207,90	69,3	3	2	1	0,5	0,45	0,54747	0,0079	69,3	57,0	NO	38,4	C348	1220,67	18,31	30,52	YES	YES
PP349	203,40	67,8	3	2	1	0,5	0,45	0,48816	0,0072	67,8	62,5	NO	42,2	C349	1379,15	20,69	34,48	YES	YES
PP364	415,50	69,3	3	2	2	0,5	0,45	0,65095	0,0094	138,5	47,9	NO	64,6	C364	1313,72	19,71	32,84	YES	YES
PP365	426,90	71,2	3	2	2	0,5	0,45	0,66881	0,0094	142,3	47,9	NO	64,6	C365	1331,39	19,97	33,28	YES	YES
PP366	544,50	90,8	3	2	2	0,5	0,45	0,807675	0,0089	181,5	50,6	NO	68,3	C366	1667,45	25,01	41,69	YES	YES
PP367	570,00	95,0	3	2	2	0,5	0,45	0,7885	0,0083	190	54,2	NO	73,2	C367	2080,79	31,21	52,02	YES	YES
PP368	467,40	77,9	3	2	2	0,5	0,45	0,73226	0,0094	155,8	47,9	NO	64,6	C368	2069,69	31,05	51,74	YES	YES
PP371	173,10	57,7	3	2	1	0,5	0,45	0,61162	0,0106	57,7	42,5	NO	28,7	C371	823,60	12,35	20,59	YES	YES
PP373	241,50	40,3	3	2	2	0,5	0,45	0,527275	0,0131	80,5	34,4	NO	46,4	C373	927,53	13,91	23,19	YES	YES
PP374	271,80	45,3	3	2	2	0,5	0,45	0,59343	0,0131	90,6	34,4	NO	46,4	C374	1327,03	19,91	33,18	YES	YES
PP375	263,40	87,8	3	2	1	0,5	0,45	0,53558	0,0061	87,8	73,8	NO	49,8	C375	1437,72	21,57	35,94	YES	YES
PP376	275,70	91,9	3	2	1	0,5	0,45	0,56059	0,0061	91,9	73,8	NO	49,8	C376	1532,68	22,99	38,32	YES	YES
PP377	258,60	86,2	3	2	1	0,5	0,45	0,52582	0,0061	86,2	73,8	NO	49,8	C377	1403,01	21,05	35,08	YES	YES
PP378	217,50	72,5	3	2	1	0,5	0,45	0,44225	0,0061	72,5	73,8	SI	49,8	C378	1201,72	18,03	30,04	YES	YES
PP392	263,40	87,8	3	2	1	0,5	0,45	0,439	0,005	87,8	90,0	SI	60,7	C392	1983,11	29,75	49,58	YES	YES
PP393	208,20	69,4	3	2	1	0,5	0,45	0,347	0,005	69,4	90,0	SI	57,6	C393	2006,79	30,10	50,17	YES	YES
PP396	218,40	72,8	3	2	1	0,5	0,45	0,30576	0,0042	72,8	107,1	SI	64,9	C396	1310,55	19,66	32,76	YES	YES
PP397	209,10	69,7	3	2	1	0,5	0,45	0,29274	0,0042	69,7	107,1	SI	63,5	C397	1469,63	22,04	36,74	YES	YES
JLL303	233,46	77,82	3	2	1	0,5	0,75	0,606996	0,0078	77,82	96,15	SI	104,2	C395	4534,77	68,02	113,37	YES	NO
JLL304	291,09	97,03	3	2	1	0,5	0,75	0,756834	0,0078	97,03	96,15	NO	108,2	C303	3830,71	57,46	95,77	YES	YES
JLL305	212,13	70,71	3	2	1	0,5	0,75	0,551538	0,0078	70,71	96,15	SI	100,6	C304	4898,39	73,48	122,46	YES	NO
JLL306	147,00	49	3	2	1	0,5	0,75	0,4557	0,0093	49	80,65	SI	76,8	C305	4518,68	67,78	112,97	YES	NO
JLL307	142,20	47,4	3	2	1	0,5	0,75	0,44082	0,0093	47,4	80,65	SI	75,3	C306	2648,99	39,73	66,22	YES	YES
JLL308	262,05	87,35	3	2	1	0,5	0,75	0,829825	0,0095	87,35	78,95	NO	88,8	C307	2749,60	41,24	68,74	YES	YES
JLL309	196,26	65,42	3	2	1	0,5	0,75	1,027094	0,0157	65,42	47,77	NO	53,7	C308	2696,07	40,44	67,40	YES	NO
JLL317	158,67	52,89	3	2	1	0,5	0,75	0,364941	0,0069	52,89	108,70	SI	90,0	C316	2190,85	32,86	54,77	YES	YES
JLL318	107,10	35,7	3	2	1	0,5	0,75	0,24633	0,0069	35,7	108,70	SI	67,1	C317	1722,98	25,84	43,07	YES	YES
JLL319	70,50	24,69	2,86	2	1	0,5	0,75	0,022221	0,0009	24,69	833,33	SI	52,1	C314	1966,46	29,50	49,16	YES	YES
JLL347	105,80	27,4	3,86	2	1	0,5	0,45	0,23838	0,0087	27,4	51,72	SI	35,0	C343	1358,71	20,38	33,97	YES	YES
JLL354	102,00	18,17	5,61	2	1	0,5	0,45	0,001817	0,0001	18,17	4500	SI	45,8	C349	1379,15	20,69	34,48	YES	YES
JLL356	83,20	56,38	1,48	2	1	0,5	0,45	0,005638	0,0001	56,38	4500	SI	37,2	C355	1178,31	17,67	29,46	YES	YES
JLL359	128,28	42,76	3	2	1	0,5	0,45	0,004276	0,0001	42,76	4500	SI	57,5	C356	1285,97	19,29	32,15	YES	YES
JLL361	281,94	93,98	3	2	1	0,5	0,45	0,009398	0,0001	93,98	4500	SI	125,5	C360	3849,28	57,74	96,23	YES	YES
JLL362	247,08	82,36	3	2	1	0,5	0,45	0,008236	0,0001	82,36	4500	SI	110,2	C362	4182,60	62,74	104,56	YES	YES
JLL368	203,13	67,71	3	2	1	0,5	0,45	0,636474	0,0094	67,71	47,87	NO	32,3	C368	2069,69	31,05	51,74	YES	NO
JLL379	64,98	21,66	3	2	1	0,5	0,45	0,017328	0,0008	21,66	562,5	SI	28,7	C379	607,06	9,11	15,18	YES	YES
JLL381	190,14	63,38	3	2	1	0,5	0,45	0,050704	0,0008	63,38	562,5	SI	80,7	C380	2370,36	35,56	59,26	YES	YES
JLL382	245,04	81,68	3	2	1	0,5	0,45	0,24504	0,003	81,68	150	SI	80,2	C382	2481,29	37,22	62,03	YES	YES
JLL383	294,27	98,09	3	2	1	0,5	0,45	0,29427	0,003	98,09	150	SI	89,1	C382	2481,29	37,22	62,03	YES	YES
JLL384	280,74	93,58	3	2	1	0,5	0,45	0,28074	0,003	93,58	150	SI	86,9	C383	2491,98	37,38	62,30	YES	YES
JLL385	180,48	60,16	3	2	1	0,5	0,45	0,18048	0,003	60,16	150	SI	64,9	C384	2266,74	34,00	56,67	YES	YES
JLL386	206,49	68,83	3	2	1	0,5	0,45	0,247788	0,0036	68,83	125	SI	67,3	C385	2424,81	36,37	60,62	YES	YES
JLL387	240,09	80,03	3	2	1	0,5	0,45	0,288108	0,0036	80,03	125	SI	73,5	C318/C385	3641,78	54,63	91,04	YES	NO
JLL388	140,34	46,78	3	2	1	0,5	0,45	0,168408	0,0036	46,78	125	SI	51,3	C387	1677,75	25,17	41,94	YES	YES
JLL394	510,32	63,79	3	2	1	0,5	0,45	0,051032	0,0008	63,79	562,5	SI	81,2	C394	4379,44	65,69	109,49	YES	NO
JLL395	620,10	68,9	3	2	1	0,5	0,45	0,05512	0,0008	68,9	562,5	SI	87,3	C394	4379,44	65,69	109,49	YES	NO

4.5 Alternative 4. SuDS solution with stormwater tank

Alternative 4 is based on the results of alternative 3. The development of this alternative will be carried out if the third alternative does not achieve the discharge flow in a natural state, offering an alternative that has a better hydraulic response to the problem.

In the third alternative, it is difficult to know the discharge flow in the design phase because it is complex to determine the response of each sustainable drainage system and it would not be possible to really reproduce its behaviour. For this reason, there are modelling tools dedicated to this, which consider many influential parameters in the behaviour of SuDS, such as the SWMM program.

This alternative will be developed based on the sustainable drainage systems of alternative 3 and the design of the storm tank of alternative 2 to reach the target discharge flow threshold of the study.

From a hydraulic and hydrological point of view, there is no doubt that this solution will be the one with the best hydrological and hydraulic response. However, there are many more factors that depend on the decision making of the solution to be developed.

4.5.1 Layout

Since we cannot design the storm tank without a discharge flow, the whole design process of this alternative will be shown in the following modelling chapter, after the results of alternative 3 have been obtained. Below is the layout of alternative 4 developed from the modelling results.

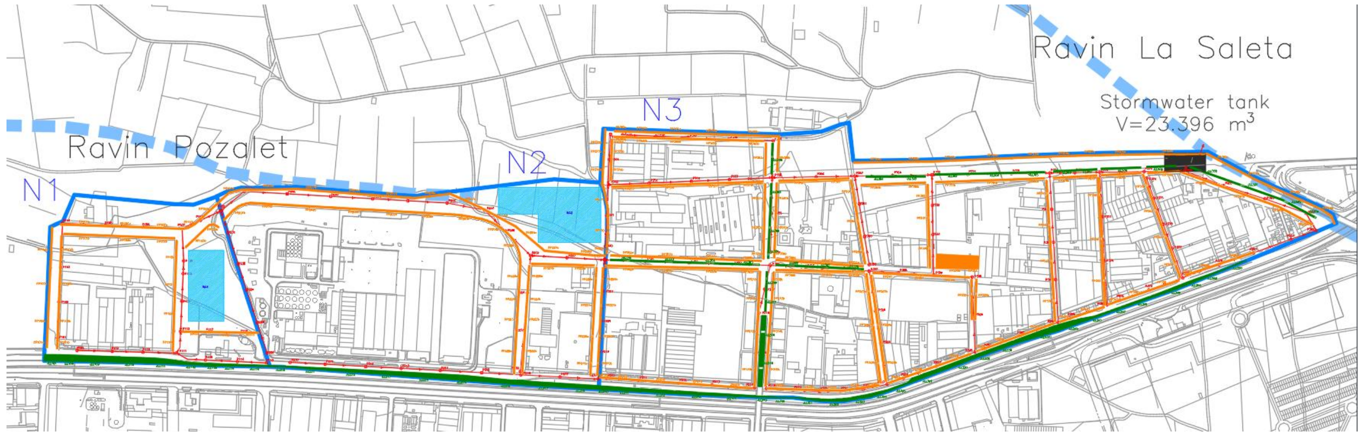


Figure 38. Alternative 4 layout

5 MODELLING OF SOLUTIONS PROPOSAL

5.1 Mathematical modelling of urban drainage

Mathematical models of a physical system are a simplification of reality that preserves the main characteristics. They are made up of:

- **Inputs:** the data of the real physical system that must be measured and to which the system responds and evolves. (rainfall)
- **Parameters:** System parameters are stationary properties of the system that are assumed to be known. They are concerned with characterizing the physical system under analysis. (Roughness, runoff threshold).
- **Equations**: mathematical expressions that describe the processes under analysis and the relationships between variables and between variables and parameters.
- **State variables:** these are numerical values which, unlike the parameters, vary over time and define the state of the system at any given moment. (Flow rates and water level).

Mathematical modelling of urban drainage tends to represent the processes of production of runoff from rainfall, transport of runoff on the surface and propagation of runoff in the system of collectors.

The aim of using a mathematical model for urban drainage is to obtain a response to rainfall with the following possible points to be analysed:

- Carrying out a diagnosis of the existing drainage system
- Verification of the design of a new drainage system or rehabilitation of an existing system.
- Optimisation of designs by traditional methods
- Real-time operations on an existing system.

In the case of our study, the use of a mathematical model is mainly for the design of a new system based on dimensioning calculations and, if necessary, to optimize this design.

Some mathematical models allow calibration, i.e. the modification of previously estimated values by comparing observed and simulated state variables. The program used for this study does not allow the calibration option and therefore it will not appear in the model.

5.2 Modelling theorical basis

5.2.1 Modelling of hydraulic processes

Hydraulic design is influenced by:

- **The Saint-Venant equations:** these are the equations governing one-dimensional transitory flow in laminar open flow.
- **The continuity equation:** which establishes the mass balance in each control element.
- **The quantity of motion equation:** which establishes the dynamic equilibrium of the control element.

Therefore, our hydraulic dimensioning will be induced by the same fundamental hypotheses as the Saint-Venant equations:

- One-dimensional flow according to the direction of the pipeline axis
- Uniform velocity distribution
- Smooth variation of the open surface
- Distribution of hydrostatic pressures vertically
- Small slope
- Friction losses in transitory flow comparable to those in steady flow
- Prismatic channel, with rectilinear layout, constant longitudinal geometric slope and constant cross-section, in geometry and roughness characteristics
- Incompressible flow

The system posed by the above equations can be solved numerically by three types of methods:

- Finite element methods
- Characteristic method
- Finite difference method

The programme used in this study, SWMM, which will be discussed later, uses the explicit finite difference method. The finite difference method is the most widely used among urban drainage modelling programs. This method is further divided into two types depending on whether its numerical schemes are implicit or explicit.

The basis of the finite difference method consists of discretising the two-dimensional space (x,t) into a mesh defined from a Δx and a Δt and approximating the partial derivatives of the equations. Whether a finite difference method is explicit or implicit depends on whether the process of finding the solution over time is done point-by-point on the spatial discretisation grid of the domain, or by simultaneously solving all points of the grid at each instant.

Explicit finite difference methods produce for each node as many equations as unknowns. Explicit schemes have the disadvantage of requiring very small time increments in the computational process to meet the Courant stability condition and are therefore more computationally expensive than implicit methods, although this disadvantage is mitigated when the flow is rapidly varied.

Explicit finite difference methods produce for each node as many equations as unknowns. Explicit schemes have the disadvantage of requiring very small time increments in the computational process to meet the Courant stability condition and are therefore more computationally expensive than implicit methods, although this disadvantage is mitigated when the flow is rapidly varied.

In addition, in order to be able to tackle the resolution of the pressure flow while still using the Saint-Venant free laminar equations, the concept of the Preissman slot is introduced, which makes all the closed conduits work in free laminar and not enter into pressure loss. This slot causes the closed duct to be considered as a duct opened by the key through a thin slot. The dimensions of the slot are very small compared to the cross-section of the pipe, so that the continuity is not disturbed and the charge level can be determined by the level reached in the slot.

5.3 Model parameters

This section describes the parameters chosen with respect to the hydrological and hydraulic model within the SWMM programme, commenting on those parameters that have been considered necessary to explain.

5.3.1 Infiltration model

The modelling of hydrological processes can be done by means of different models. The model used for the study is the empirical SCS (Soil Conservation Service) model of the curve number (CN).

The SCS model is suitable for the modelling of runoff production in an urban environment because:

- It respects the runoff threshold concept whereby there is no production until the runoff threshold is exceeded.
- Production tends to the value of rainfall when P tends to infinity, which in urban environments is completely true.
- In intermediate ranges, the runoff coefficient (E/P ratio) is increasing with the rainfall value.
- The fact that, for practical purposes, it only depends on one parameter (P0), makes it particularly attractive for inclusion in the complete drainage model.

In addition to the CN model, the SWMM programme offers four other models. The Horton, Modified Horton, Green and Ampt and Modified Green and Ampt models. Although these last four models reproduce much more accurately the processes involved in the production of runoff, they have the weighty disadvantage for use in urban drainage models that they depend on too many parameters, which are sometimes difficult to estimate. For this reason, the CN model was chosen for modelling infiltration.

The first parameter used is the precipitated rainfall input, assuming that, in the same catchment, different rainfall will cause different runoff. In our case, this would be the long 70-minute hietogram, although modelling has been carried out for the three proposed hietograms. Once the amount of precipitated water is known, the runoff can be obtained using the equation:

$$E = \frac{(I - 0{,}2 * S)^2}{I + 0{,}8 * S}$$

where:

- E (mm) is the runoff generated.
- I (mm) is the amount of rainfall
- S (dimensionless) is the maximum potential difference between rainfall and runoff generated.

The SCS model parameter (either S or P0) is related to the original model parameter of the curve number (CN). This index represents the combined influence of the soil aspects that determine infiltration.

The CN varies between 0 and 100, the last value corresponding to 100% runoff. The relationship between the CN and the parameters S and P0 is as follows:

$$S = \frac{25400}{CN} - 254$$

$$P_0 = \frac{5080}{CN} - 50{,}8$$

The value of the input CN curve number in the equation is obtained from tables and depends on the soil conditions, which are: land use, conservation measures used in cultivation, soil compaction in relation to land use, soil infiltration capacity and previous soil humidity condition.

The curve numbers of the SCS model have been chosen as indicated in the SWMM user manual (Rossman L. A., 2015), taking into account the type of soil and the hydrographic zone in which we find ourselves, being a type C zone as indicated previously in *Annex Nº2: Initial data and design conditions*.

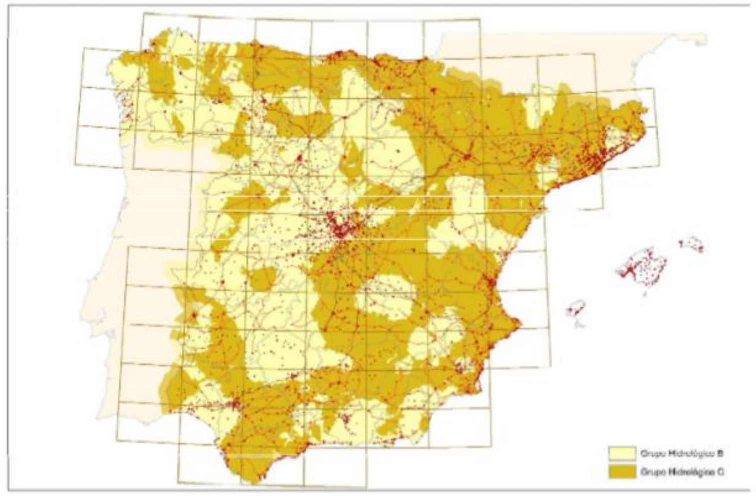


Figure 39. Hydrological land groups map. Source: Ministerio de fomento, 2016

Having made an initial separation of three soil types by giving each a runoff coefficient, this same separation has been used to determine the curve number.

One aspect to be taken into account with the curve number is the preconditions of soil humidity. The response of the soil to a rainfall event will depend to a large extent on its previous humidity status. If there has been a previous rainfall event, it is logical to assume that the soil will accept less water and the runoff will be correspondingly higher. Therefore, the curve number method considers 3 possible previous humidity states (Ibáñez, S. et al., 2011).

Condition II is the typical wetness condition at the occurrence of maximum annual runoff and this is the one shown in the tables. In addition to this data, transformations of the curve number can be found in the case of a more favourable humidity value, this being condition I, and a higher humidity value for condition III.

In the case of the study area, it is more coherent to choose condition II since it is unlikely that extreme events generally find the soil with average humidity conditions.

Conversión del número de curva de la condición II a las condiciones I y III

Número de curva en la condición	Número de curva correspondiente a las condiciones	
II	I	III
100	100	100
95	87	99
90	78	98
85	70	97
80	63	94
75	57	91
70	51	87
65	45	83
60	40	79
55	35	75
50	31	70
45	27	65
40	23	60
35	19	55
30	15	50
25	12	45
20	9	39
15	7	33
10	4	26
5	2	17
0	0	0

Figure 40. Conversion CN according to humidity conditions. Source: Ibáñez, S. et al, 2011

The transformation of the curve number for the most unfavourable humidity case has been calculated from the conversion tables and finally:

Table 29. Curve number for each zone

Zone	CN – Cond. II	CN – Cond. III
Impermeable – Streets and roadways	98	100
Edification – Private industrial areas	91	98
Permeable – Parks and green areas	74	90

However, as indicated above, it will be modelled for humidity conditions type II.

5.3.2 Hydraulic model

The SWMM software offers three hydraulic models to solve the equations:

- Uniform flow
- Kinematic wave
- Dynamic wave

Uniform flow is the simplest way to represent the behaviour of water inside conduits. This type of hydraulic model cannot take into account water storage, hydraulic overtopping, manhole inflow and outflow losses, reverse flow or pressurised flow phenomena. The use of this model has been discarded because it is only suitable for preliminary analyses using continuous simulations at large time scales.

The kinematic wave hydraulic model solves the continuity equation together with a simplified form of the equation of quantity of motion under each of the conditions. This model does not consider effects such as hydraulic overtopping, manhole inlet and outlet losses, reverse or pressurised flow and its application would be restricted to branched networks only.

Finally, the hydraulic **dynamic wave** model is the most complete and sophisticated of the three. It solves full one-dimensional Saint-Venant equations and therefore generates more accurate results. This model considers the effects of storage in conduits, hydraulic overtopping, losses at manhole inlets and outlets, and reverse and pressurized flow. The disadvantage of this model is that it requires smaller time increments, in the order of 1 minute, making the simulation take longer to run. However, the study simulation does not have so many elements, conditions or parameters to make this a limitation.

The dynamic wave model in turn offers different parameters, of which the most relevant for the simulation are listed below

- Inertia parameters: with the "DAMPEN" option the inertia terms are reduced as the flow approaches critical and ignored when the flow is supercritical.
- Definition of supercritical flow: supercritical flow is defined by checking the Froude number and the slope of the water sheet simultaneously.
- Force (drive) equation: the Darcy-Weisbach equation has been chosen to calculate the friction losses during pressurised flow over the Hazen-Williams equation because it has been shown that, for large Reynolds number in large pipes, it does not meet the design requirements because it does not consider the effect of the variation of the relative roughness (R. Flechas, 2012).

5.4 Elements definitions

The drainage system has been defined in the SWMM program by means of the geometric elements offered by the program. In this section the properties and fields to be completed to define each element in the model will be named.

In the hydraulics part they are identified:

- Nodes
    - **Junctions**
    - **Outfalls**
    - Dividers
    - **Storage Units**
- Links
    - **Conduits**
    - Pumps
    - Orifices
    - Weirs
    - Outlets
- Transects
- Controls

In the hydrology part, the following are identified:

- **Rain Gages**
- **Subcatchments**
- Aquifers
- Snow Packs
- Unit Hydrographs
- **LID Controls (SuDS)**

In the modelling of the case study, the options highlighted in bold will be used: Junctions, outfalls, conduits, rain gages, subcatchments and LID controls. Next, the elements used to model the system are identified one by one.

5.4.1 Subcatchments

Subcatchments are hydrological land units whose topography and elements of the drainage system convey runoff directly to a discharge point.

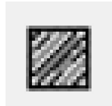


Figure 41. Identifying symbol for subcatchments in SWMM

Catchments can be further divided into permeable sub-areas and impermeable sub-areas. Surface runoff can infiltrate into the topsoil of the permeable sub-areas, but not through the impervious sub-areas.

The infiltration of rainfall in the permeable areas of a given catchment over the unsaturated topsoil can be described using three different models, as indicated above, with the SCS Curve Number model being the one of choice.

Table 30. Subcatchment properties

Concept	Definition
Name	User-assigned subcatchment name
X-Coordinate	Horizontal location of the subcatchment's centroid on the Study Area Map
Y-Coordinate	Vertical location of the subcatchment's centroid on the Study Area Map
Description	Optional description of the subcatchment.
Tag	Optional label used to categorize or classify the subcatchment
Rain Gage	Name of the rain gage associated with the subcatchment
Outlet	Name of the node or subcatchment which receives the subcatchment's runoff
Area	Area of the subcatchment, including any LID controls
Width	Area of the subcatchment. including any LID controls
% Slope	Average percent slope of the subcatchment.
% Imperv	Percent of land area (excluding the area used for LID controls) which is impervious
N-Imperv	Manning's n for overland flow over the impervious portion of the subcatchment
N-perv	Manning's n for overland flow over the pervious portion of the
Dstore-Imperv	Depth of depression storage on the impervious portion of the subcatchment
Dstore-Perv	Depth of depression storage on the pervious portion of the subcatchment
% Zero-Imperv	Percent of the impervious area with no depression storage

Subarea Routing	Choice of internal routing of runoff between pervious and impervious areas: - IMPERV: runoff from pervious area flows to impervious area - PERV: runoff from impervious area flows to pervious area - OUTLET: runoff from both areas flows directly to outlet
Percent Routed	Percent of runoff routed between subareas
Infiltration	Percent of runoff routed between subareas
LID Controls	Low impact development controls in the subcatchment
Groundwater	Groundwater flow parameters for the subcatchment
Land Uses	Name of snow pack parameter set assigned to the subcatchment
Snow Pack	Land uses to the subcatchment
Initial Buildup	Initial quantities of pollutant buildup over the subcatchment
Curb Length	Total length of curbs in the subcatchment

Each subcatchment has been identified with a code starting with a letter followed by a number. This letter indicates the type of subcatchment according to its runoff coefficient value and the first number indicates, in the case of streets and private subcatchments, the catchment to which it belongs.

- C: Street subcatchment	CXXX
- PR: Private subcatchment	PRXXX
- VE: Green area	VEXX

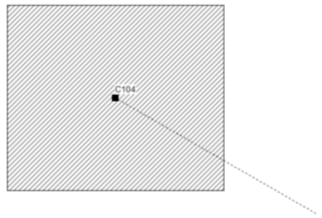


Figure 42. Graphical representation of subcatchment in SWMM

5.4.2 Junctions

Junctions are nodes in the drainage system where the different lines are connected to each other.

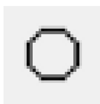


Figure 43. Identifying symbol for junctions in SWMM

Physically they can represent the confluence of natural surface channels, manholes of the drainage system or pipe connection elements. In our case all junctions are manholes.

Excess water in a junction results in a partially pressurised flow as long as the connected pipes are under charge. This excess water can be completely lost from the system or pond at the top and then re-enter the connection.

Table 31. Junctions properties

Concept	Definition
Name	User-assigned junction name
X-Coordinate	Horizontal location of the junction on the Study Area Map
Y-Coordinate	Vertical location of the junction on the Study Area Map
Description	Optional description of the junction
Tag	Optional label used to categorize or classify the junction
Inflows	External direct, dry weather or RDII inflows to the junction
Treatment	Set of treatment functions for pollutants entering the node
Invert El.	Invert elevation of the junction
Max. Depth	Maximum depth of junction
Initial Depth	Depth of water at the junction at the start of the simulation
Surcharge Depth	Additional depth of water beyond the maximum depth that is allowed before the junction floods
Ponded Area	Area occupied by ponded water atop the junction after flooding occurs

Each junction representing the manholes has been identified with a code where the beginning is the letter P to indicate that it is a manhole and it is followed by three numbers where the first one is the number of the catchment where it is located.

PXXX

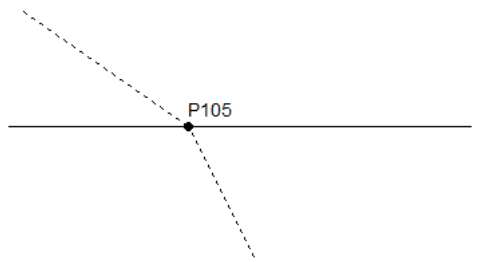


Figure 44. Graphical representation of junction in SWMM

5.4.3 Outfall

Outfalls are end nodes of the drainage system used to define the final downstream boundary conditions of the system in the case of using the dynamic wave flow model.

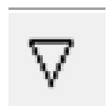


Figure 45. Symbol representing outfall in SWMM

Table 32. Junction properties

Concept	Definition
Name	User-assigned outfall name
X-Coordinate	Horizontal location of the outfall on the Study Area Map
Y-Coordinate	Vertical location of the outfall on the Study Area Map
Description	Optional description of the outfal
Tag	Optional label used to categorize or classify the outfall
Inflows	External direct, dry weather or RDII inflows to the outfall
Treatment	Set of treatment functions for pollutants entering the node
Invert El.	Invert elevation of the outfall
Tide Gate	Presence of tide gate: - YES - tide gate present to prevent backflow - NO - no tide gate present
Route To	Optional name of a subcatchment that receives the outfall's discharge
Type	Type of outfall boundary condition: - FREE: outfall stage determined by minimum of critical flow depth and normal flow depth in the connecting conduit - NORMAL: outfall stage based on normal flow depth in connecting conduit - FIXED: outfall stage set to a fixed value - TIDAL: outfall stage given by a table of tide elevation versus time of day - TIMESERIES: outfall stage supplied from a time series of elevations
Fixed Stage	Water elevation for a FIXED type of outfall
Tidal Curve Name	Name of the Tidal Curve relating water elevation to hour of the day for a TIDAL outfall
Time Series Name	Name of time series containing time history of outfall elevations for a TIMESERIES outfall

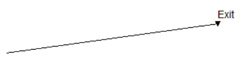


Figure 46. Graphical representation of outfall in SWMM

5.4.4 Storage Unit

Storage units are nodes in the drainage system with the capacity to store certain volumes of water.

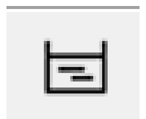


Figure 47. Symbol representing storage units in SWMM

Physically, they can represent from small storage systems such as small basins to large systems such as lakes. In the study storage units are used as storm tanks.

Table 33. Storage Unit properties

Concept	Definition
Name	User-assigned storage unit name
X-Coordinate	Horizontal location of the storage unit on the Study Area Map
Y-Coordinate	Vertical location of the storage unit on the Study Area Map
Description	Optional description of the storage unit
Tag	Optional label used to categorize or classify the storage unit
Inflows	External direct, dry weather or RDII inflows to the storage unit
Treatment	Set of treatment functions for pollutants entering the node
Invert El.	Elevation of the bottom of the storage unit
Max. Depth	Elevation of the bottom of the storage unit
Initial Depth	Initial depth of water in the storage unit at the start of the simulation
Evap. Factor	The fraction of the potential evaporation from the storage unit's water surface that is actually realized
Seepage Loss	Optional soil properties that determine seepage loss through the bottom and sloped sides of the storage unit
Storage Loss	Method of describing how the surface area of the storage unit varies with water depth: - FUNCTIONAL uses the function $Area = A * (Depth)^B + C$ to describe how surface area varies with depth; - TABULAR uses a tabulated area versus depth curve. In either case, depth is measured in feet above the bottom and surface area
FUNCTIONAL	
Coeff.	A-value in the functional relationship between surface area and storage depth
Exponent	B-value in the functional relationship between surface area and storage depth
Constant	C-value in the functional relationship between surface area and storage depth
TABULAR	
Curve Name	Name of the Storage Curve containing the relationship between surface area and storage depth

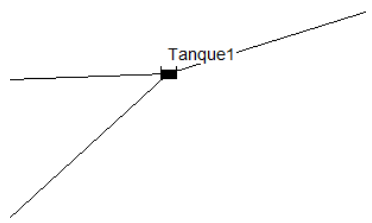


Figure 48. Graphical representation storage unit in SWMM

5.4.5 Conduits

Conduits are pipes or canals through which water moves from one node of the conveyance system to another.

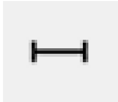


Figure 49. Representative symbol conduits in SWMM

It is possible to select the cross-section of different varieties of geometries, both open and closed. In the case study all the conduits are circular collectors of various diameters.

Table 34. Conduits properties

Concept	Definition
Name	User-assigned conduit name
Inlet Node	Name of node on the inlet end of the conduit
Outlet Node	Name of node on the outlet end of the conduit
Description	Optional description of the conduit
Tag	Optional label used to categorize or classify the conduit
Shape	Geometric properties of the conduit's cross section
Max. Depth	Maximum depth of the conduit's cross section
Length	Conduit length
Roughness	Manning's roughness coefficient
Inlet Offset	Depth or elevation of the conduit invert above the node invert at the upstream end of the conduit
Outlet Offset	Depth or elevation of the conduit invert above the node invert at the downstream end of the conduit
Initial Flow	Depth or elevation of the conduit invert above the node invert at the downstream end of the conduit
Maximum Flow	Maximum flow allowed in the conduit
Entry Loss Coeff.	Head loss coefficient associated with energy losses at the entrance of the conduit
Exit Loss Coeff.	Head loss coefficient associated with energy losses at the exit of the conduit
Avg. Loss Coeff.	Head loss coefficient associated with energy losses along the length of the conduit
Flap Gate	Presence of flap gate: - YES if a flap gate exists that prevents backflow through the conduit - NO if no flap gate exists
Culvert Code	Code number of inlet geometry if conduit is a culvert

Each conduit has a code starting with L, indicating that it is a linear collector, followed by six numbers separated in half by a dash. The numbers indicate the inlet and outlet manhole code.

LXXX-XXX

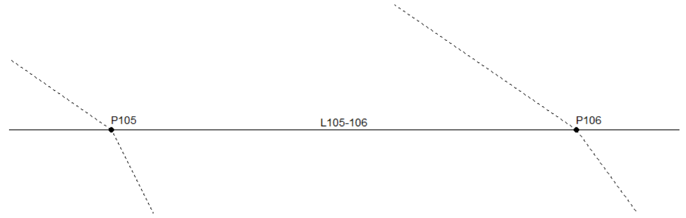


Figure 50. Graphical representation of conduits in SWMM

5.4.6 Rain Gages

Rain Gages provide the input data for rainfall occurring over one or more of the defined catchments in the study area.

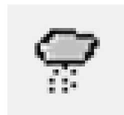


Figure 51. Rain gages representative symbol in SWMM

The rainfall of the study areas is the same for all subcatchments as the study area is small and affects everywhere the same rainfall. The rainfall data can be defined by time series data or come from a file external to the program.

For the case study, rain gage with three time series is proposed, corresponding to the short-, medium- and long-term design storm.

Table 35. Rain gage properties

Concept	Definition
Name	User-assigned rain gage name
X-Coordinate	Horizontal location of the rain gage on the Study Area Map
Y-Coordinate	Vertical location of the rain gage on the Study Area Map
Description	Optional description of the rain gage
Tag	Edit an optional description of the rain gage
Rain Format	Format in which the rain data are supplied: - INTENSITY: each rainfall value is an average rate in inches/hour (or mm/hour) over the recording interval, - VOLUME: each rainfall value is the volume of rain that fell in the recording interval (in inches or millimetres), - CUMULATIVE: each rainfall value represents the cumulative rainfall that has occurred since the start of the last series of non-zero values (in inches or millimetres).
Rain Interval	Recording time interval between gage readings in either decimal hours or hours:minutes format.
Snow Catch Factor	Factor that corrects gage readings for snowfall.
Data Source	Source of rainfall data; either TIMESERIES for user-supplied time series data or FILE for an external data file

TIME SOURCE	
Series Name	Name of time series with rainfall data
DATA FILE	
File Name	Name of external file containing rainfall data
Station No.	Name of external file containing rainfall data
Rain Units	Depth units for rainfall values in user-prepared files

5.4.7 SuDS modelling

The modelling of sustainable drainage systems, called LID (Low Impact Development) in the program, is carried out according to the following conceptual scheme where, depending on the SuDS typology, the element will be composed of different layers and flows.

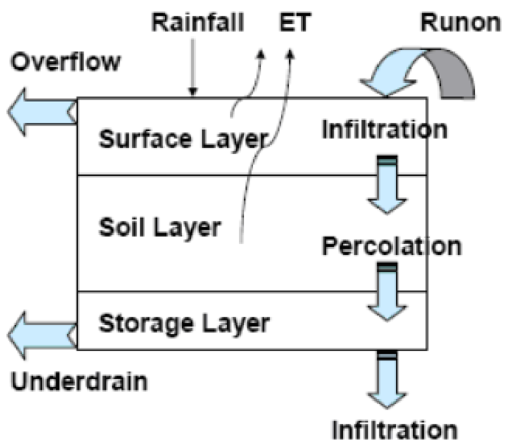


Figure 52. Conceptual diagram of SuDS representation. Source: Rossman et al., 2015

The different possible layers in the program are:
- **Surface layer:** which corresponds to the ground or pavement surface that receives direct rainfall and runon from upstream land areas, stores excess inflow in depression storage, and generates surface outflow that either enters the drainage system or flows onto downstream land areas.
- **Soil layer:** which is the engineered soil mixture used in bio-retention cells to support vegetative growth. It can also be a sand layer placed beneath a pavement layer to provide bedding and filtration.
- **Storage layer:** which is a bed of crushed rock or gravel that provides storage in bio-retention cells, porous pavement, and infiltration trench systems. For a rain barrel it is simply the barrel itself.

The input flows in the diagram are two:
- Rainfall
- Runon

The internal flows between the layers correspond to:
- Infiltration between surface and soil layer
- Percolation between soil and storage layer

Finally, the outflows of the diagram are:
- Overflow
- Underdrain
- Infiltration between storage and the ground
- Evapotranspiration

Depending on the type of SUDS to be implemented, the program offers the following combinations of diagrams representing the schemes of each typology:

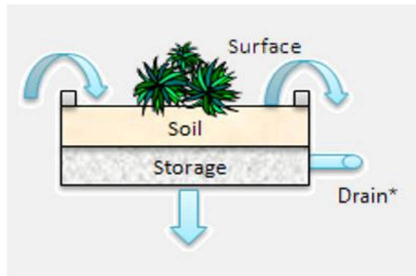


Figure 53. Bio-retention cell scheme. Source: SWMM

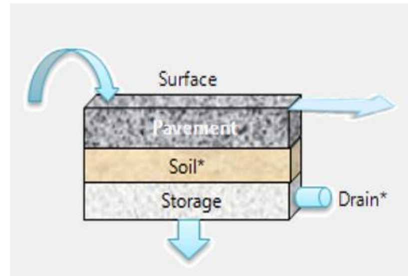


Figure 54. Pervious pavement scheme. Source: SWMM

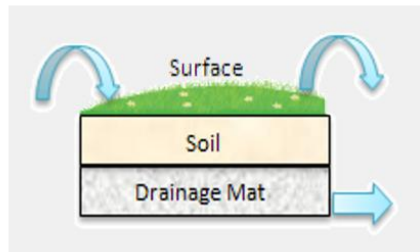


Figure 55. Green roof scheme. Source: SWMM

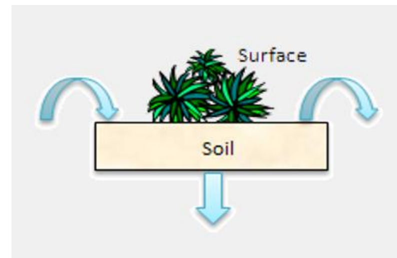


Figure 56. Rain garden scheme. Source: SWMM

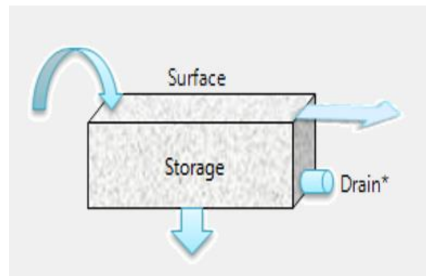


Figure 57. Infiltration trench scheme. Source: SWMM

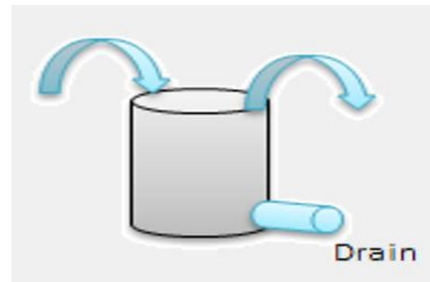


Figure 58. Rain barrel scheme. Source: SWMM

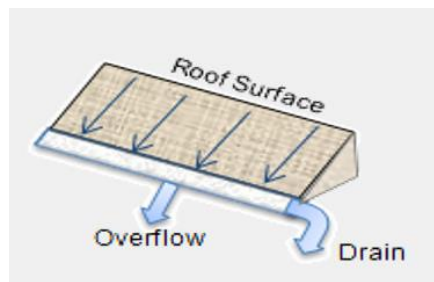


Figure 59. Rooftop connection scheme. Source: SWMM

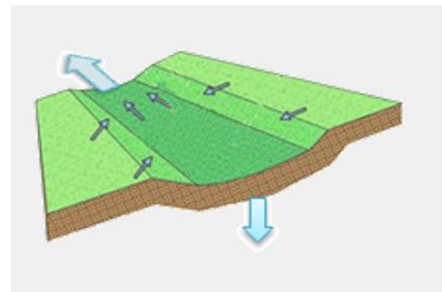


Figure 60. Vegetative swale scheme. Source: SWMM

Thus, the combination of layering possibilities of the different proposed SuDS will be:

Table 36. Layers used to model different types of LID units

LID type	Surface	Pavement	Soil	Storage	Drain	Drainage Mat
Bio-Retention Cell	x		x	Optional	Optional	
Rain Garden	x		x			
Green Roof	x		x			x
Permeable Pavement	x	x	Optional	x	Optional	
Infiltration Trench	x			x	Optional	
Rain Barrel				x	x	
Roof Disconnection	x				x	
Vegetative Swale	x					

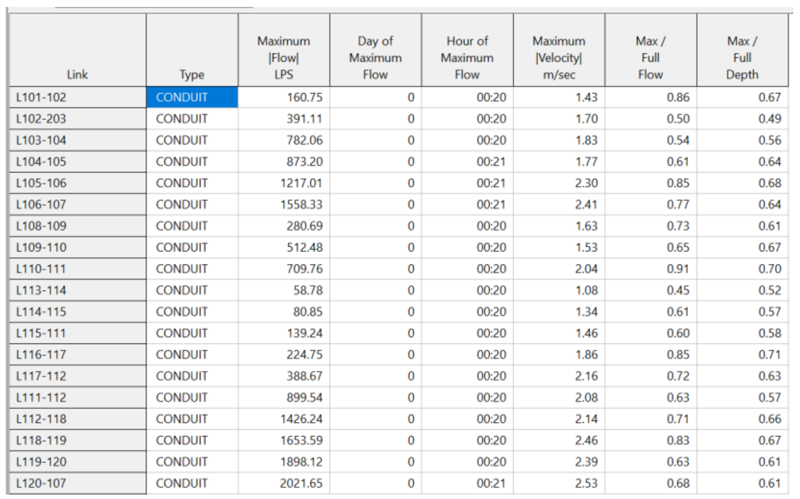
In the modelling of alternative 2 it will be specified which SUDS have been used and the chosen layer parameters.

5.5 Interpretation of modelling results

The SWMM programme offers different ways of interpreting the results, either with tables showing the maximum results or the results at each time interval, with graphs showing the maximum value for each instant or visual simulations of the time-varying network.

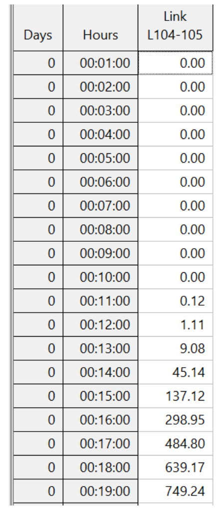
For the analysis of the results, a mixture of all the proposed results has been used.

The tables give the exact value of any parameter we are looking for either in maximum value or each value per time interval. In the following image you can see an example:



Link	Type	Maximum |Flow| LPS	Day of Maximum Flow	Hour of Maximum Flow	Maximum |Velocity| m/sec	Max / Full Flow	Max / Full Depth
L101-102	CONDUIT	160.75	0	00:20	1.43	0.86	0.67
L102-203	CONDUIT	391.11	0	00:20	1.70	0.50	0.49
L103-104	CONDUIT	782.06	0	00:20	1.83	0.54	0.56
L104-105	CONDUIT	873.20	0	00:21	1.77	0.61	0.64
L105-106	CONDUIT	1217.01	0	00:21	2.30	0.85	0.68
L106-107	CONDUIT	1558.33	0	00:21	2.41	0.77	0.64
L108-109	CONDUIT	280.69	0	00:20	1.63	0.73	0.61
L109-110	CONDUIT	512.48	0	00:20	1.53	0.65	0.67
L110-111	CONDUIT	709.76	0	00:20	2.04	0.91	0.70
L113-114	CONDUIT	58.78	0	00:20	1.08	0.45	0.52
L114-115	CONDUIT	80.85	0	00:20	1.34	0.61	0.57
L115-111	CONDUIT	139.24	0	00:20	1.46	0.60	0.58
L116-117	CONDUIT	224.75	0	00:20	1.86	0.85	0.71
L117-112	CONDUIT	388.67	0	00:20	2.16	0.72	0.63
L111-112	CONDUIT	899.54	0	00:20	2.08	0.63	0.57
L112-118	CONDUIT	1426.24	0	00:20	2.14	0.71	0.66
L118-119	CONDUIT	1653.59	0	00:20	2.46	0.83	0.67
L119-120	CONDUIT	1898.12	0	00:20	2.39	0.63	0.61
L120-107	CONDUIT	2021.65	0	00:21	2.53	0.68	0.61

Figure 61. Example of maximum parameters related with link flow for each conduit. Source: SWMM



Days	Hours	Link L104-105
0	00:01:00	0.00
0	00:02:00	0.00
0	00:03:00	0.00
0	00:04:00	0.00
0	00:05:00	0.00
0	00:06:00	0.00
0	00:07:00	0.00
0	00:08:00	0.00
0	00:09:00	0.00
0	00:10:00	0.00
0	00:11:00	0.12
0	00:12:00	1.11
0	00:13:00	9.08
0	00:14:00	45.14
0	00:15:00	137.12
0	00:16:00	298.95
0	00:17:00	484.80
0	00:18:00	639.17
0	00:19:00	749.24

Figure 62. Example of link flow minutal variation on a conduit. Source: SWMM

The graphs are used to quickly interpret the hydrological and hydraulic behaviour of the system and to identify anomalous results, an example of which is shown below:

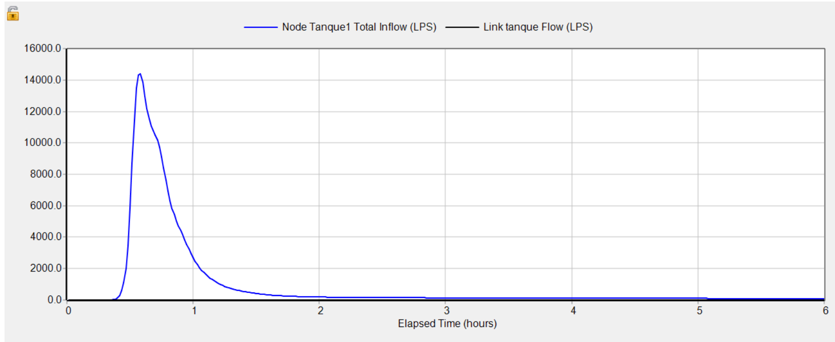


Figure 63. Example representation of peak flow in a conduit during the design storm. Source: SWMM

Finally, the visual simulations allow not only to know the hydraulic behaviour, but also to view the system of the collector network in general, allowing to visually identify if any error has been made when entering the geometric data of the solution.

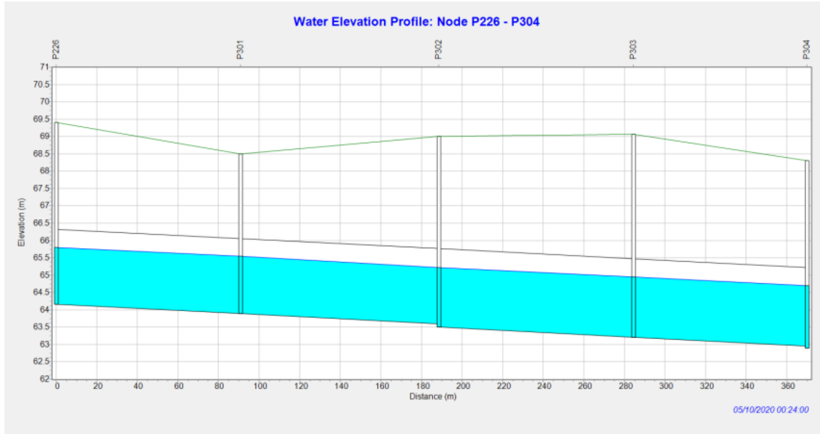


Figure 64. Example of flow variation in an instant. Source: SWMM

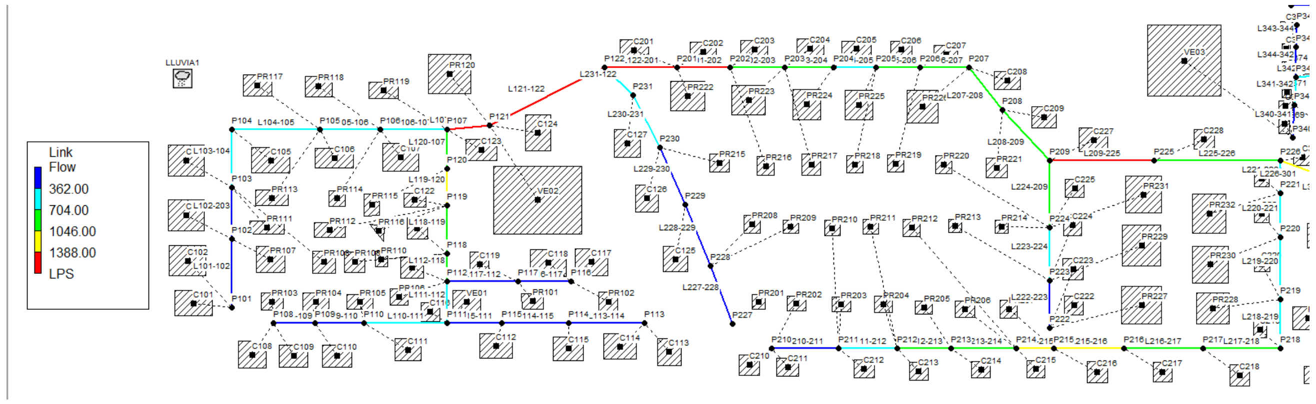


Figure 65. Example of flow variation in the conduits in an instant by means of colour coding. Source: SWMM

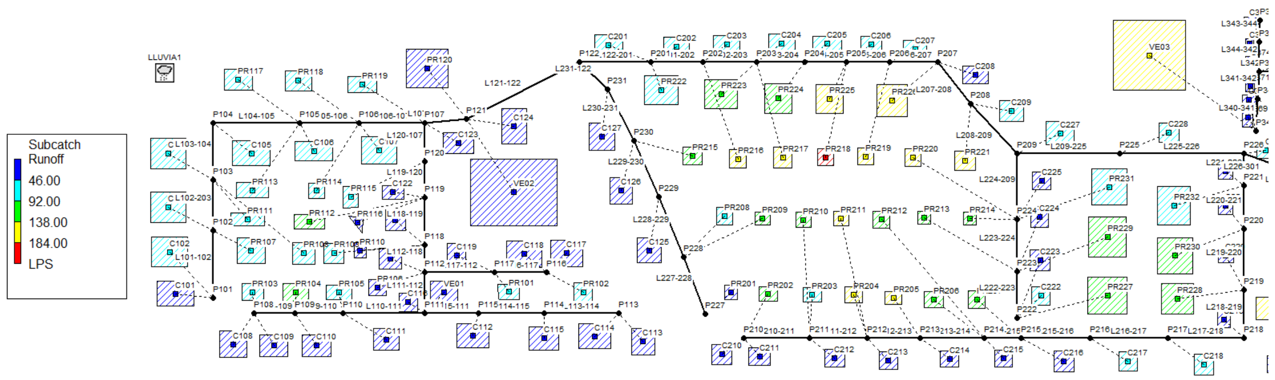


Figure 66. Example of runoff production variation by subcatchment at a given instant. Source: SWMM

5.6 Modelling of alternative 1. Conventional drainage system

The modelling of scenario 1 is based on the previously calculated design. It should be noted that the design has been calculated with respect to the IDF curve. *In Annex Nº2: Initial data and design conditions*, in the section corresponding to the pluviometric analysis, the reasons why these curves were not the most representative of the area were defined and that there were design downpours determined by the two-parameter gamma method that were closer to reality.

Therefore, the objective of the modelling is to use as a basis the results obtained in the design to verify by means of the SWMM program that this design is correct for short-, medium- and long-term design storms and, if possible, to optimize the design.

5.6.1 Modelling representation alternative 1

After listing the different elements that allow the modelling of a drainage system, this section shows the visual aspect of alternative 1 implemented in the SWMM programme.

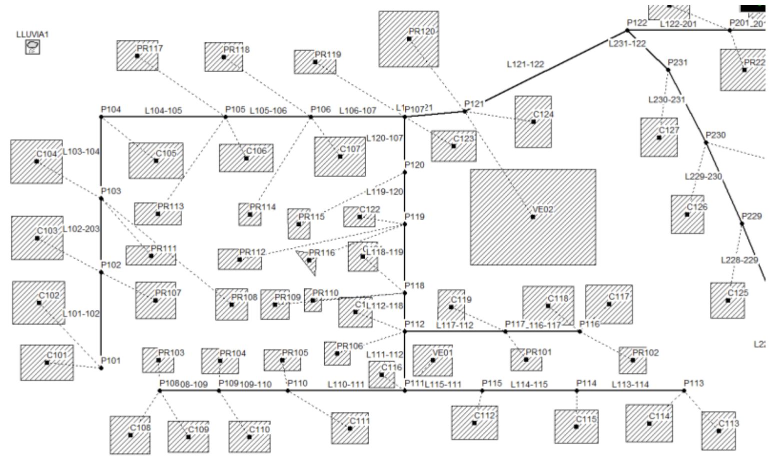


Figure 67. Modelling catchment N1. Alternative 1

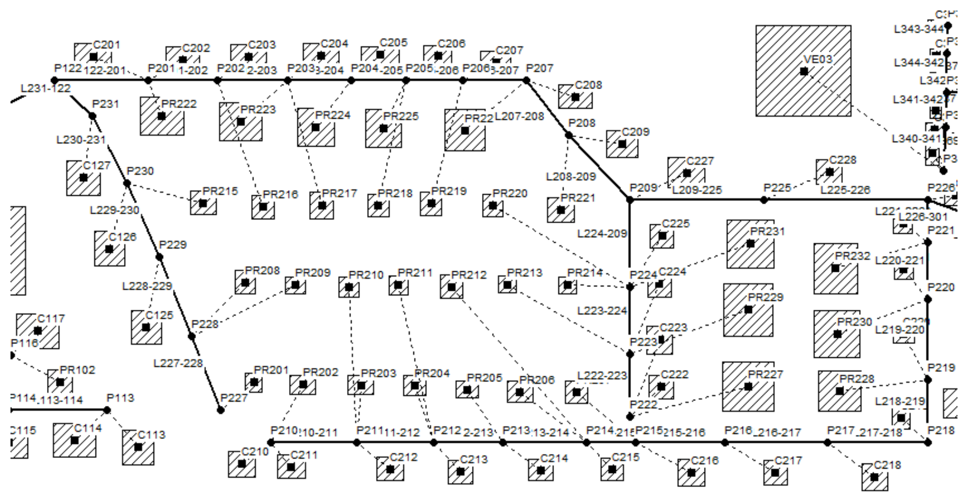


Figure 68. Modelling catchment N2. Alternative 1

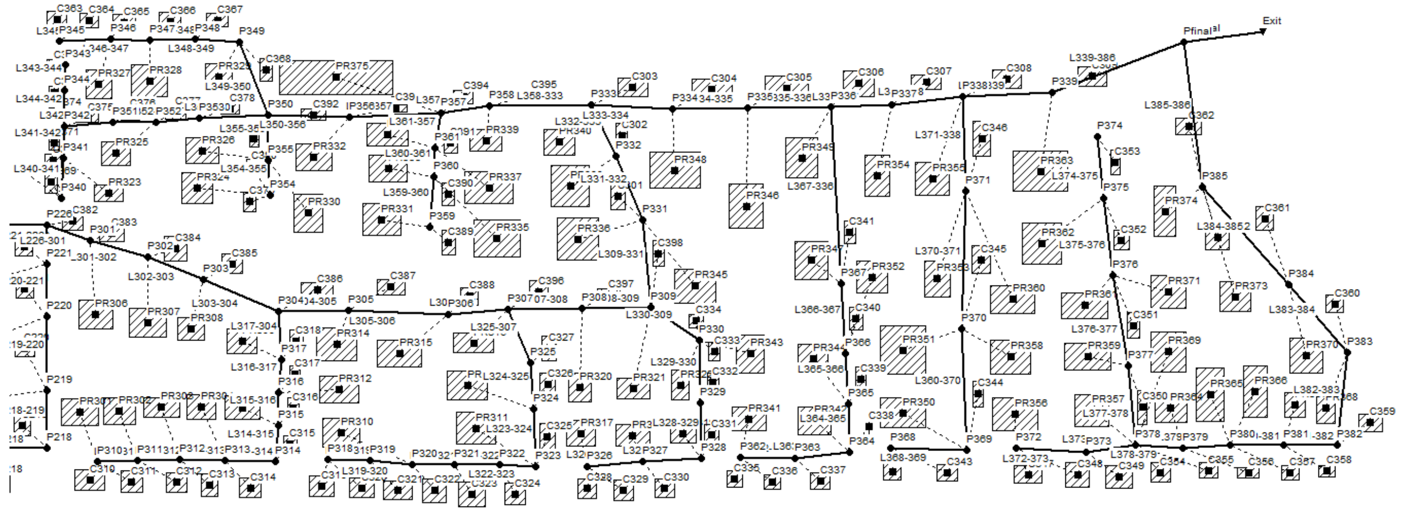


Figure 69. Modelling catchment N3. Alternative 1

5.6.2 Modelling results alternative 1

The following values at the discharge point were obtained from the subsequent simulation using the short-, medium- and long-term design storms:

Table 37. Results on discharge point. Alternative 1

Design storm	Peak flow (m3/s)	Volume	Maximum depth
	m3/s	m3	m
Short-term	24,49	25.945	2,23
Medium-term	29,71	40.208	2,43
Long-term	33,29	74.809	2,58

Knowing the results of the modelling, it was considered interesting to highlight, by means of a comparison, that the choice of the SCS Curve Number (CN) method in the modelling is much more appropriate for the study than the runoff coefficients method used in the design with the modified rational method.

To this end, the results of the volume generated by the rational method using runoff coefficients and the volume generated by the curve number method were compared in order to demonstrate that the curve number method is capable of reproducing events of any nature, regardless of their associated return period, and that the use of the rational method would be conservative for events of short duration and under-dimensioned for events of long duration.

From the calculation of the runoff coefficients, it was obtained:

Table 38. Average runoff coefficient

Catchment	Surface	Average runoff coefficient
	m2	-
N1	156.713	0,703
N2	341.245	0,716
N3	696.606	0,804
TOTAL	**1.194.565**	**0,766**

Table 39. Comparison of volumes generated by method.

Design storm	Rainfall depth	Rainfall volume	RATIONAL METHOD		MODEL	
			C,average	V,runoff	C,avergae	V,runoff
	mm	m3	-	m3	-	m3
Short-term	30,9	36.912	0,766	28.275	0,703	25.945
Medium-term	43,7	52.202	0,766	39.987	0,770	40.208
Long-term	73,6	87.920	0,766	67.347	0,851	74.809

From the results of the above table, it can be concluded that, with the rational method, the runoff volume generated does not take into account the variation of the runoff coefficient as the duration or intensity of rainfall varies. This indicates that the impervious surface is the same for any design shower analysed.

In contrast, the curve number method does take into account the variation in impervious surface area, the higher the volume of rainfall, the greater the variation in impervious surface area, thus adapting to different design rainfall with different properties.

The modelling results for alternative 1 are presented below in terms of flow, volume and draught within the conduit of the last segment of collector system.

5.6.2.1 Short-term design storm. Alternative 1

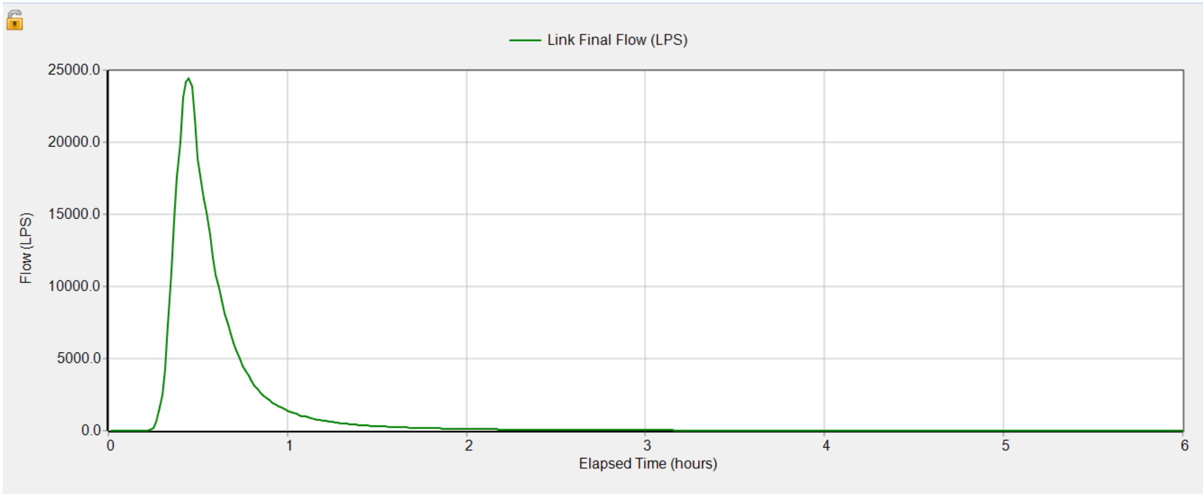


Figure 70. Discharged flow on short-term design storm. Alternative 1

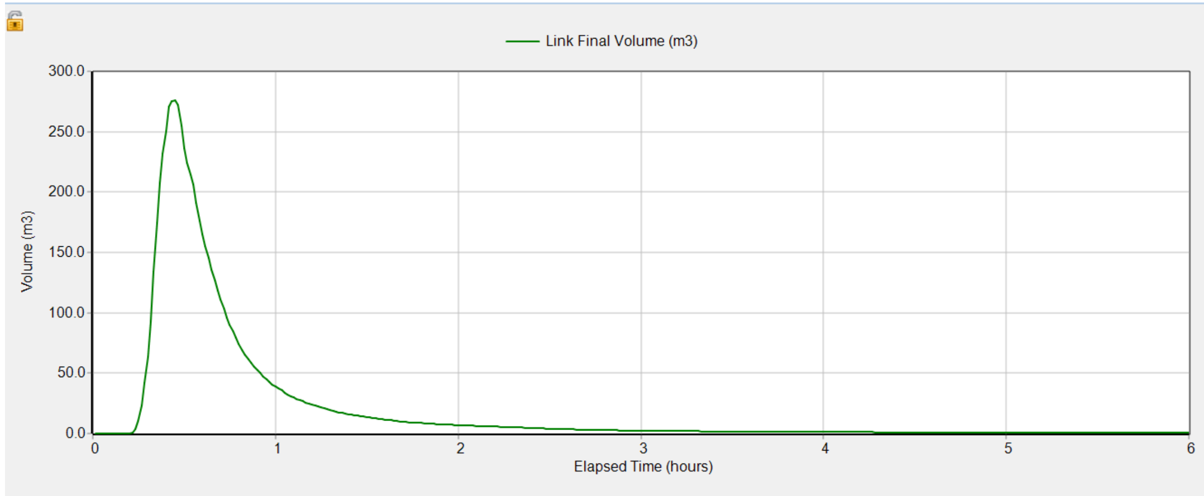


Figure 71. Discharged volume on short-term design storm. Alternative 1

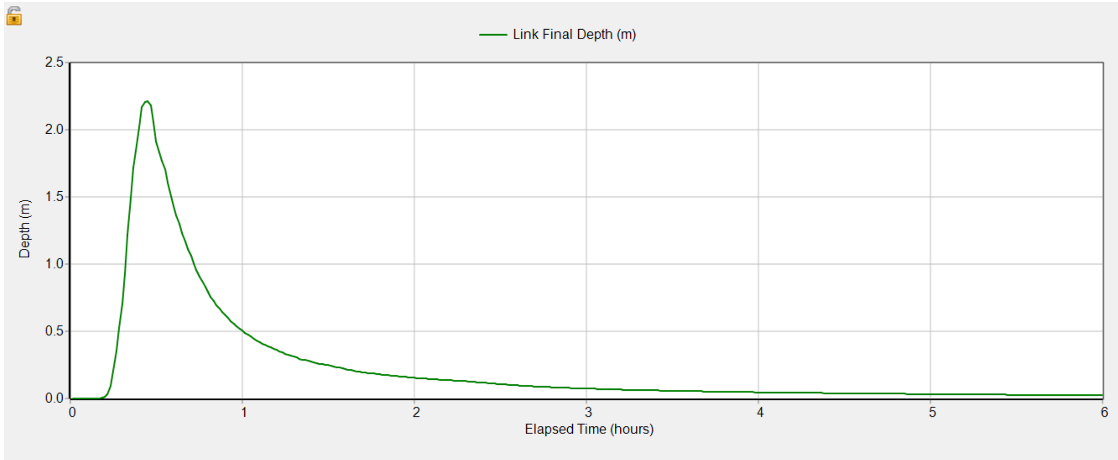


Figure 72. Depth on short-term design storm. Alternative 1

5.6.2.2 Medium-term design storm. Alternative 1

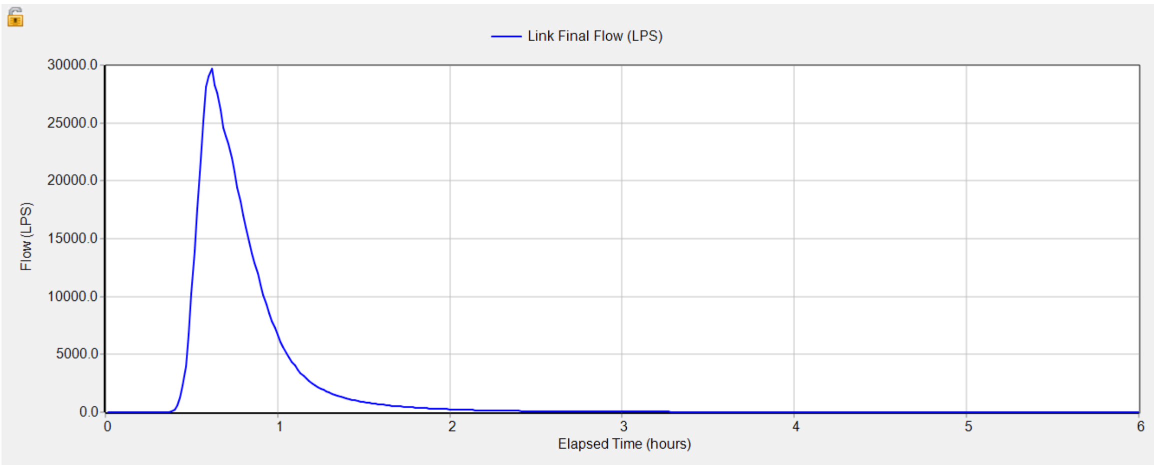


Figure 73. Discharged flow on medium-term design storm. Alternative 1

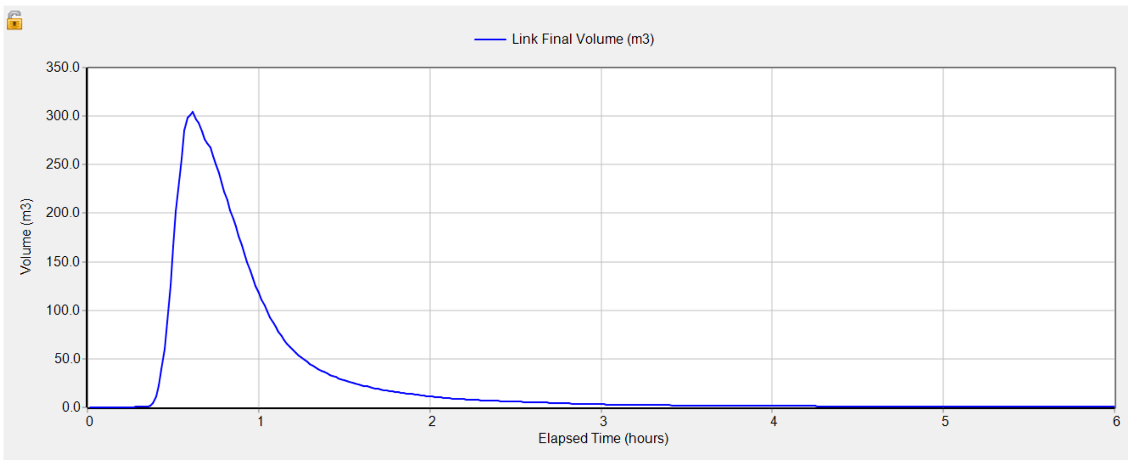


Figure 74. Figure 75. Discharged volume on medium-term design storm. Alternative 1

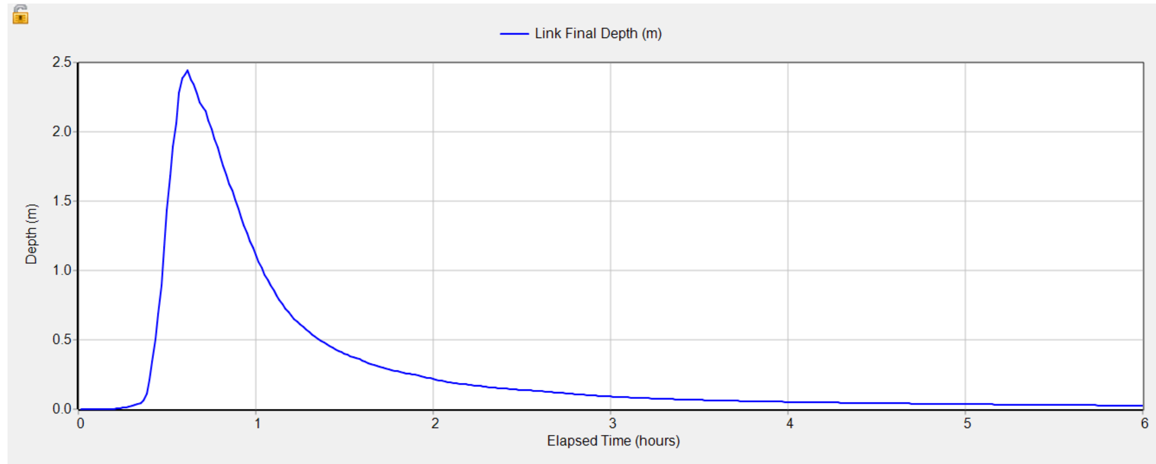


Figure 76. Depth on medium-term design storm. Alternative 1

5.6.2.3 Long-term design storm. Alternative 1

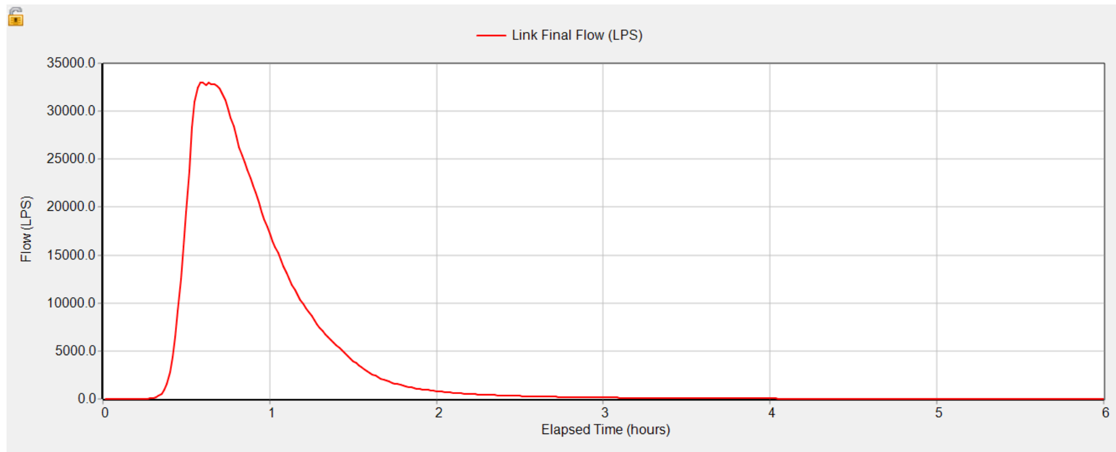


Figure 77. Discharged flow on long-term design storm. Alternative 1

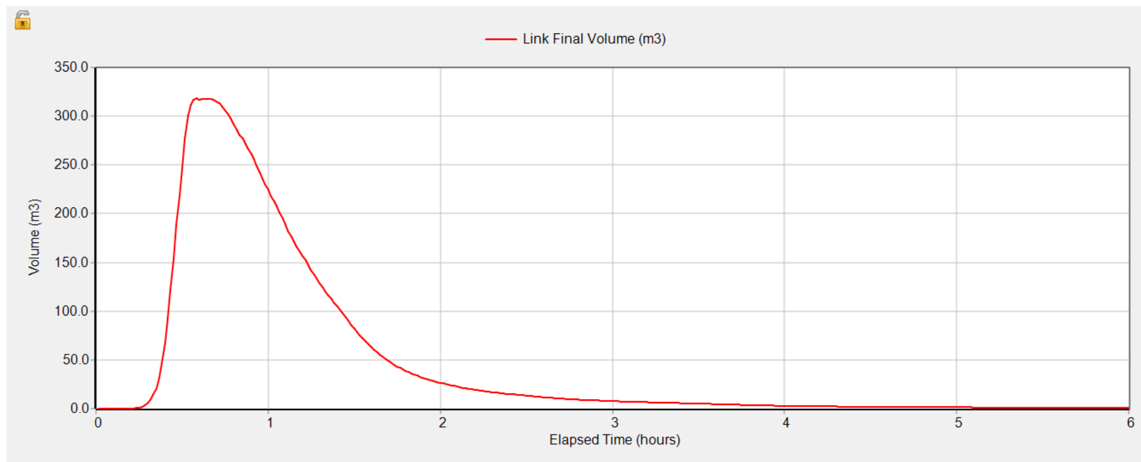


Figure 78. Discharged volume on long-term design storm. Alternative 1

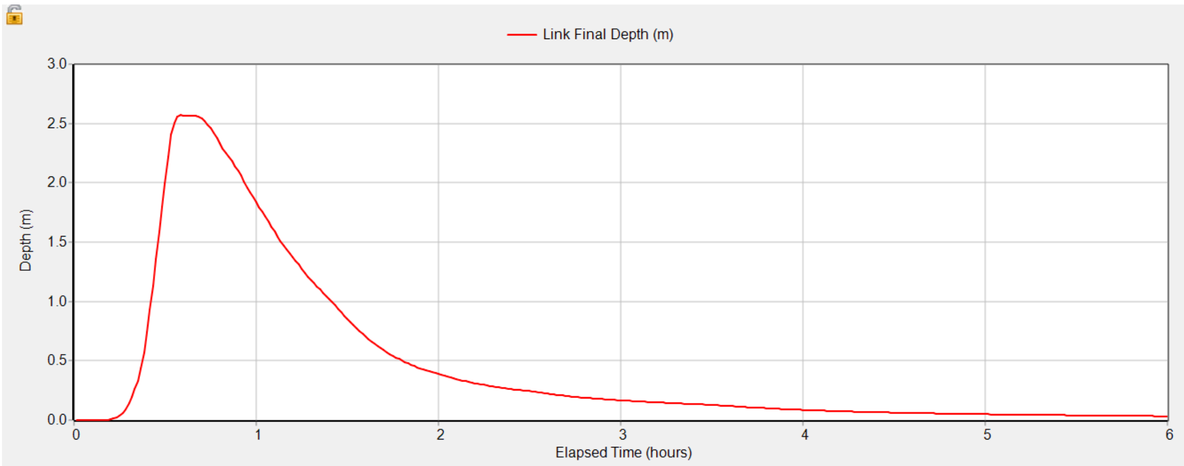


Figure 79. Depth on long-term design storm. Alternative 1

5.7 Modelling of alternative 2. Stormwater tank

This modelling is based on the previous modelling by adding a storm tank in the final section of the conventional collector system to laminate the outflow.

The objective of this section is, firstly, to model the system after the implementation of a storm tank of the dimensions calculated in the design and, secondly, to optimise the design of this storm tank to meet the natural flow objectives with the short-, medium- and long-term design storms.

The optimisation is necessary because prior to the simulation with the storm tank it can be seen that the inflow into the tank after modelling the long-term storm is higher than the inflow used for the design with the IDF curve.

5.7.1 Modelling representation of alternative 2

The representation is very similar to the representation of modelling 1 but changing the end connection node to a storage unit.

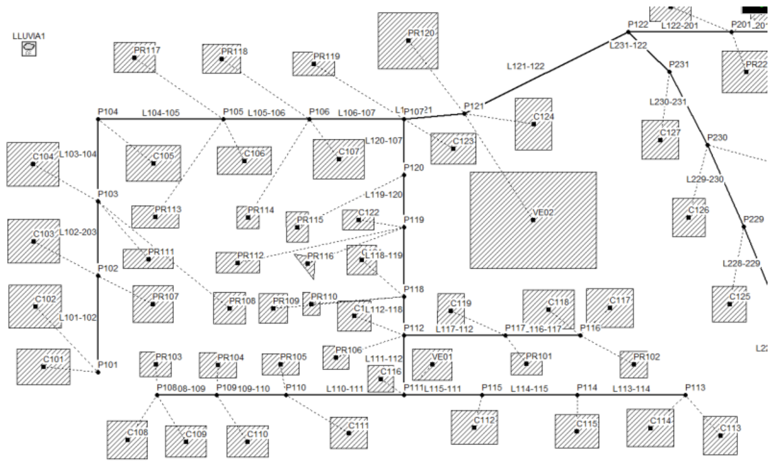


Figure 80. Modelling catchment N1. Alternative 2

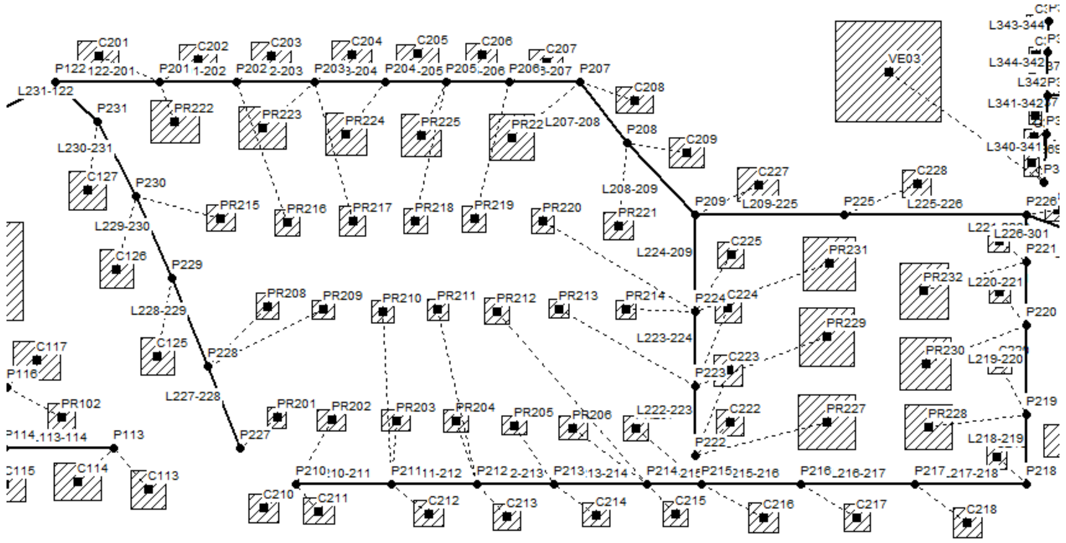


Figure 81. Modelling catchment N2. Alternative 2

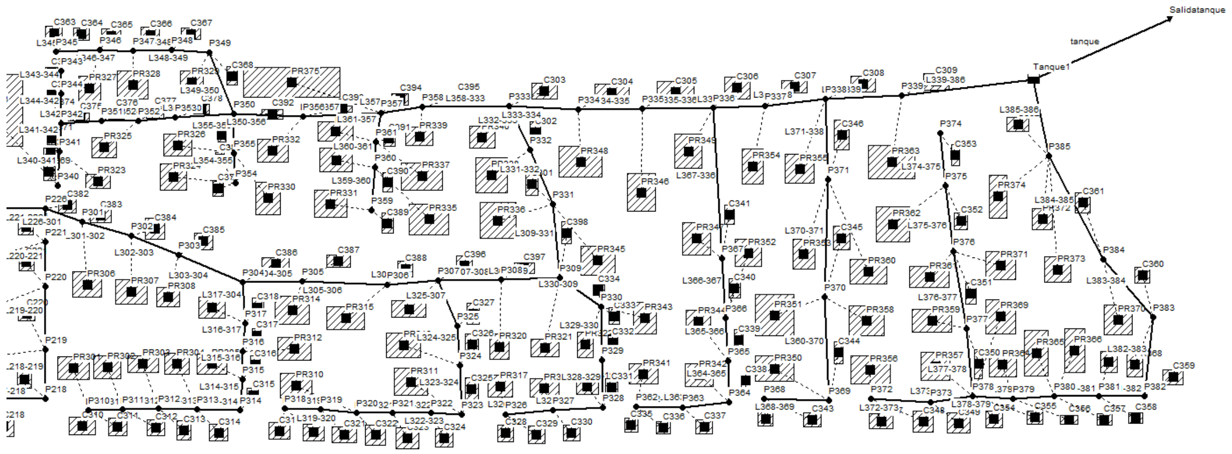


Figure 82. Modelling catchment N3. Alternative 2

5.7.2 Modelling results alternative 2

Table 40. Inflow to stormwater tank

Design storm	Peak flow (m3/s)	Volume
	m3/s	m3
Short-term	24,49	25.945
Medium-term	29,71	40.208
Long-term	33,29	74.809

With these data and the implementation of the dimensioned storm tank, the results of peak flow, volume and maximum discharge depth are obtained. In addition, it has been considered of interest to represent the depth inside the tank and the comparison of the inflow and outflow of the storm tank.

A reminder of the dimensions of the storm tank:

$$S_{tank} = \mathbf{5.160}\ \boldsymbol{m2}$$
$$h_{tank} = \mathbf{7,0}\ \boldsymbol{m}$$

Table 41. Results on discharge point. Alternative 2 without optimisation

Design storm	Peak flow (m3/s)	Volume	Maximum depth
	m3/s	m3	m
Short-term	0,00	0	0,00
Medium-term	6,89	14.325	1,01
Long-term	20,26	48.797	2,05

5.7.2.1 Short-term design storm. Alternative 2 without optimisation

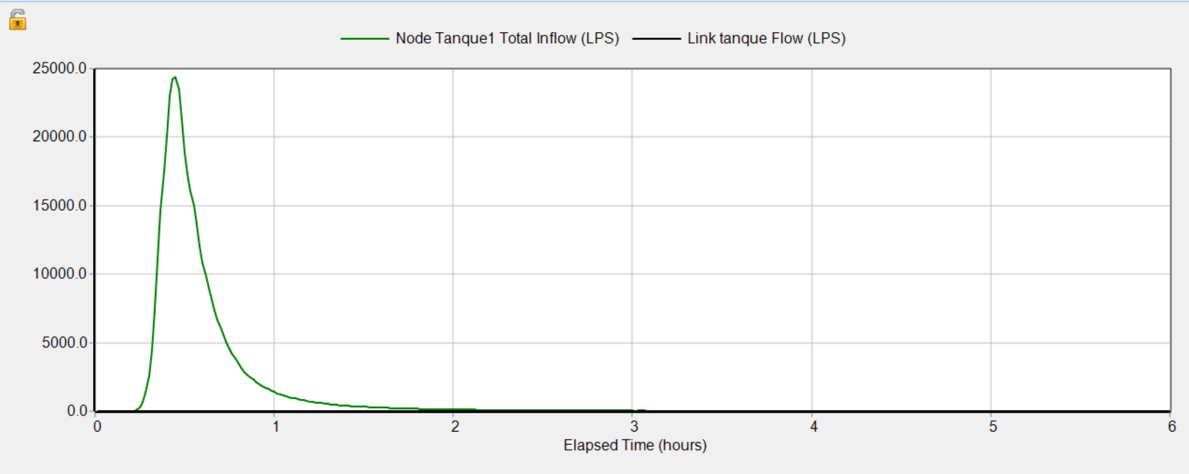


Figure 83. Comparison of inflow and outflow of the storm tank on short-term design storm. Alternative 2 without optimisation

In the case of the short-term design storm, the implementation of the storm tank means that nothing is discharged into the receiving medium. In this case the tank would be oversized for this design storm. This is not surprising since the short design shower is the most favourable of the three.

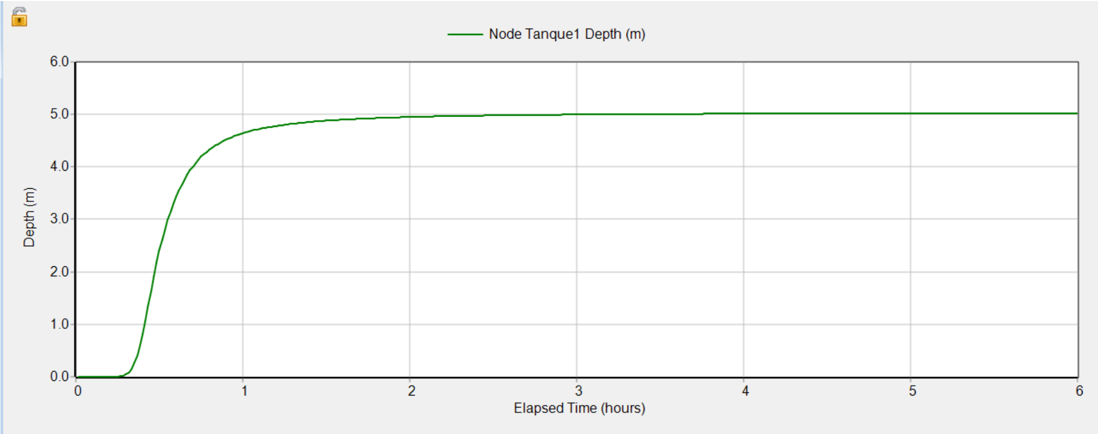


Figure 84. Inner depth of the storm tank short-term design storm. Alternative 2 without optimisation

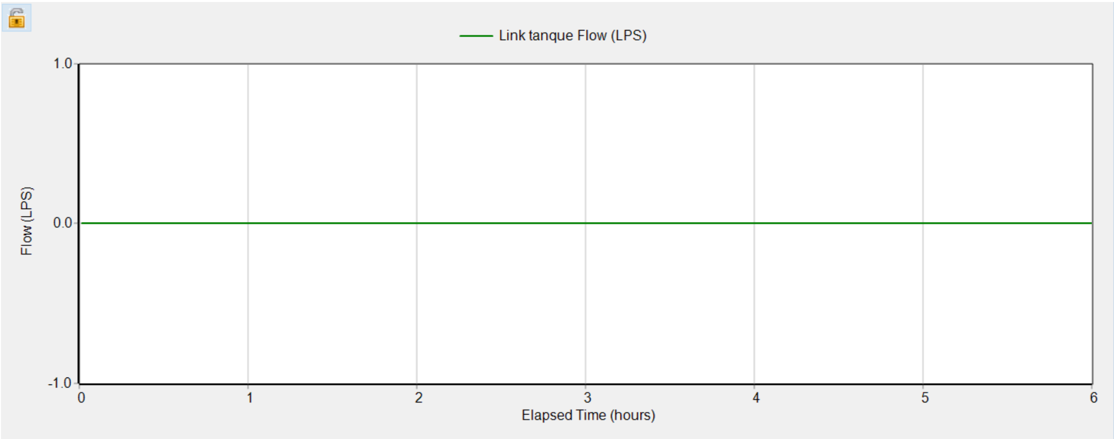


Figure 85. Discharged flow on short-term design storm. Alternative 2 without optimisation

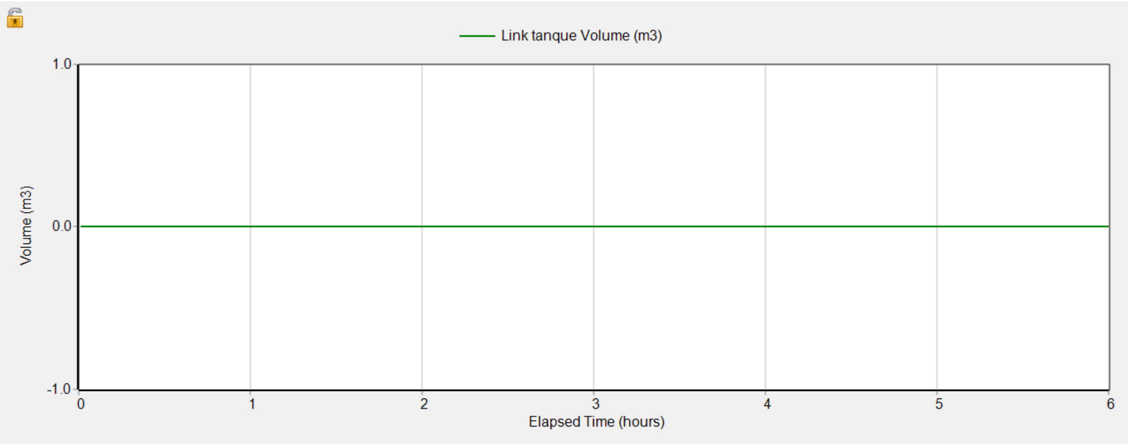


Figure 86. Discharged volume on short-term design storm. Alternative 2 without optimisation

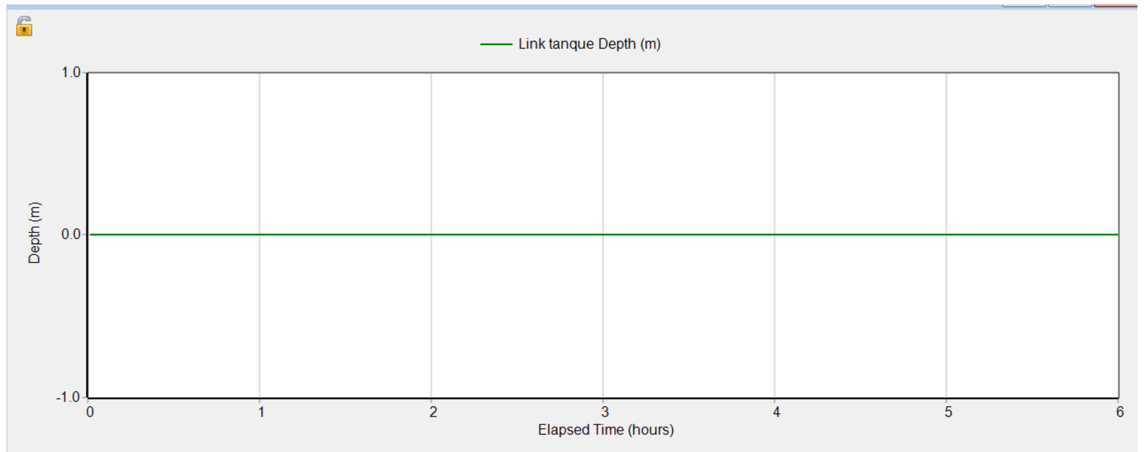


Figure 87. Depth on short-term design storm. Alternative 2 without optimisation

5.7.2.2 Medium-term design storm. Alternative 2 without optimisation

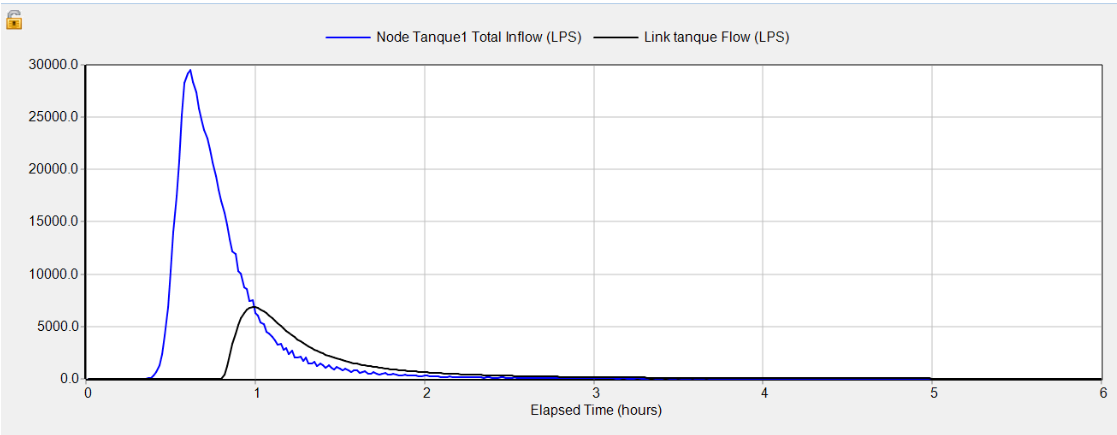


Figure 88. Comparison of inflow and outflow of the storm tank on medium-term design storm. Alternative 2 without optimisation

The comparison of both inlet and outlet hydrographs is shown in the following graph, where the inlet flow is shown in blue and the outlet flow in black.

In the case of the medium-term design storm, it can be seen that the tank has fulfilled its function of flow reduction. It also coincides that the outflow is very close to the design outflow for which the storm tank was designed.

This is great news when it comes to checking the correct design and modelling. The main reason is because the design was carried out with the IDF curve which, as shown in the rainfall analysis, was close to the medium-term design storm. The fact that the modelling result for the medium-term design storm resembles the design result for the IDF curve affirms the coherence of the results.

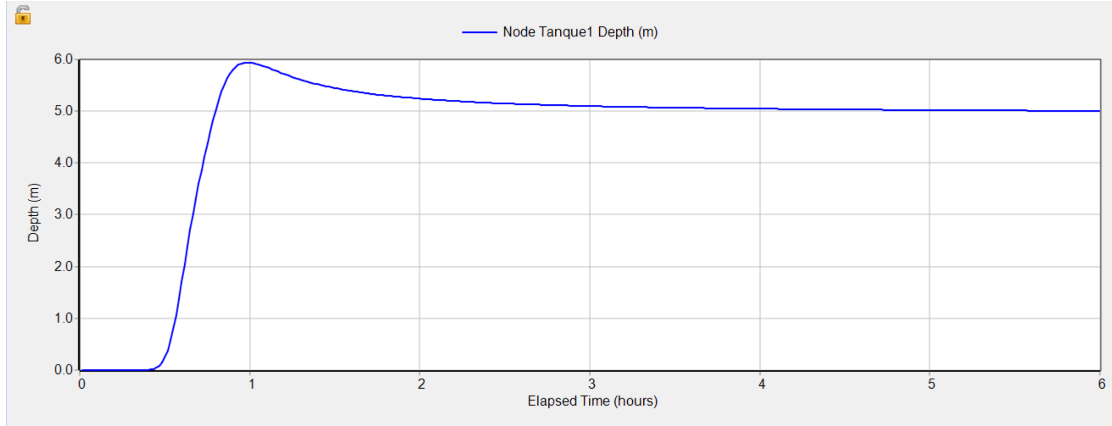


Figure 89. Inner depth of the storm tank medium-term design storm. Alternative 2 without optimisation

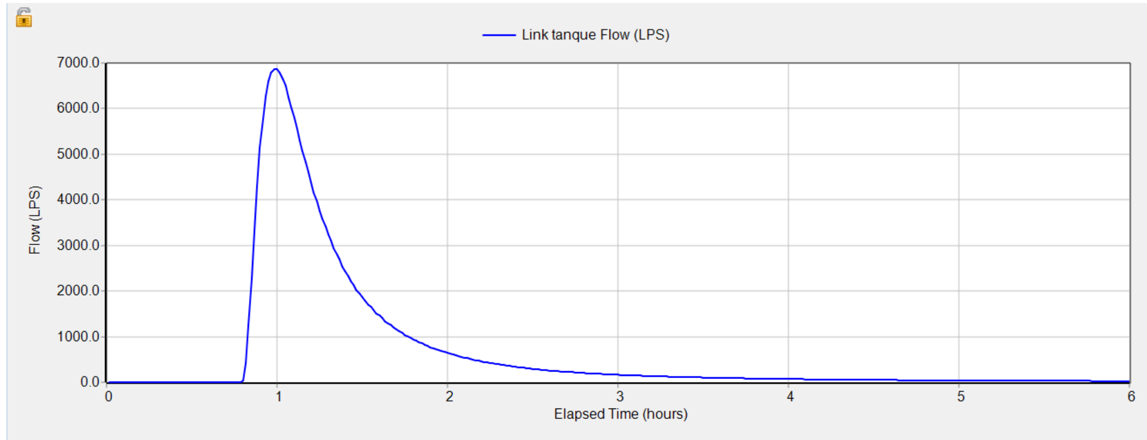


Figure 90. Discharged flow on medium-term design storm. Alternative 2 without optimisation

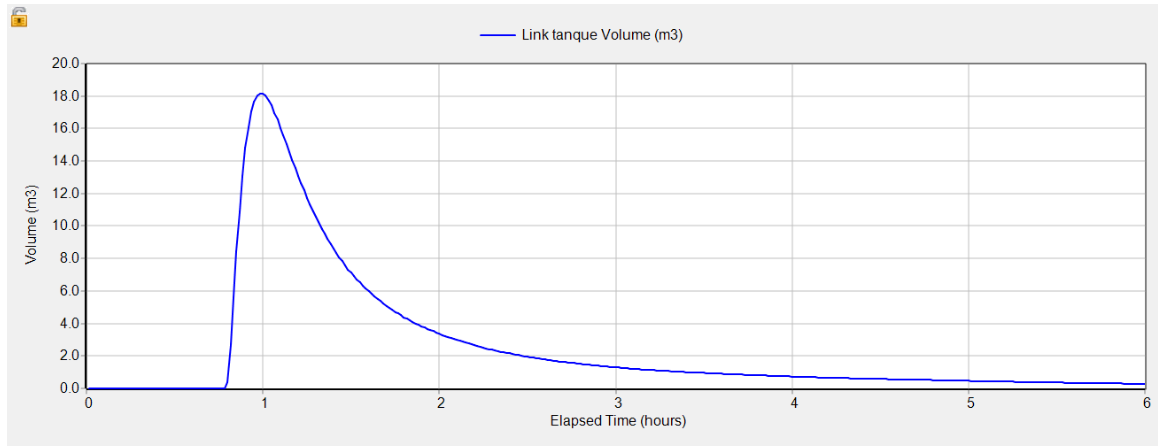


Figure 91. Discharged volume on medium-term design storm. Alternative 2 without optimisation

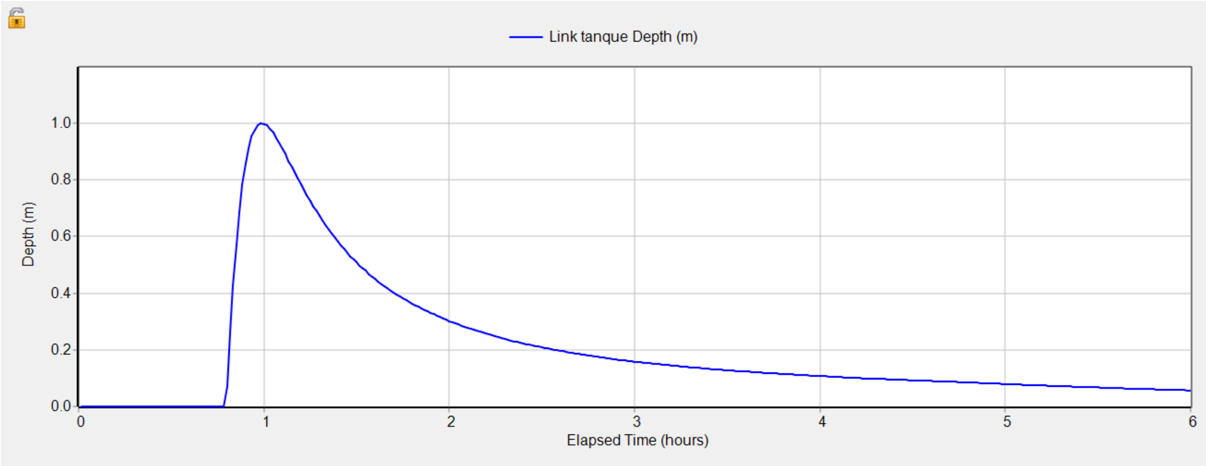


Figure 92. Depth on medium-term design storm. Alternative 2 without optimisation

5.7.2.3 Long-term design storm. Alternative 2 without optimisation

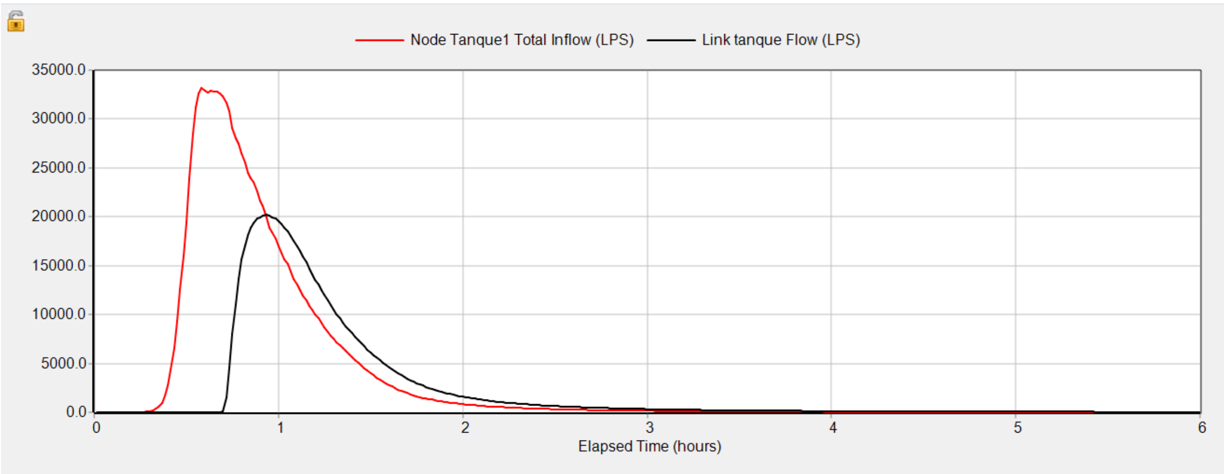


Figure 93. Comparison of inflow and outflow of the storm tank on long-term design storm. Alternative 2 without optimisation

The comparison of both inflow and outflow hydrographs is shown in the graph below, where the inflow is shown in red and the outflow in black.

As the storm tank is in series and not in parallel, this is reflected in the hydrograph by shifting the flow curve to the right. The explanation is based on the design of the tank. If the tank had been in parallel, the evolution of the flow discharge up to the natural flow would be the same as in alternative 1, this evolution would stop at the limit of the natural flow following a horizontal line in the graph and meanwhile the storm tank would be storing the excess flow. Once a flow rate equal to or below the natural flow rate is reached, the storage of water would stop and the entire inflow would correspond to the outflow.

In the case of the in series configuration, the storm tank starts storing water from the first instant, and it is only when the storage limit is reached that the tank proceeds to release the outflow. Therefore, the outflow curve is out of phase with the inflow curve.

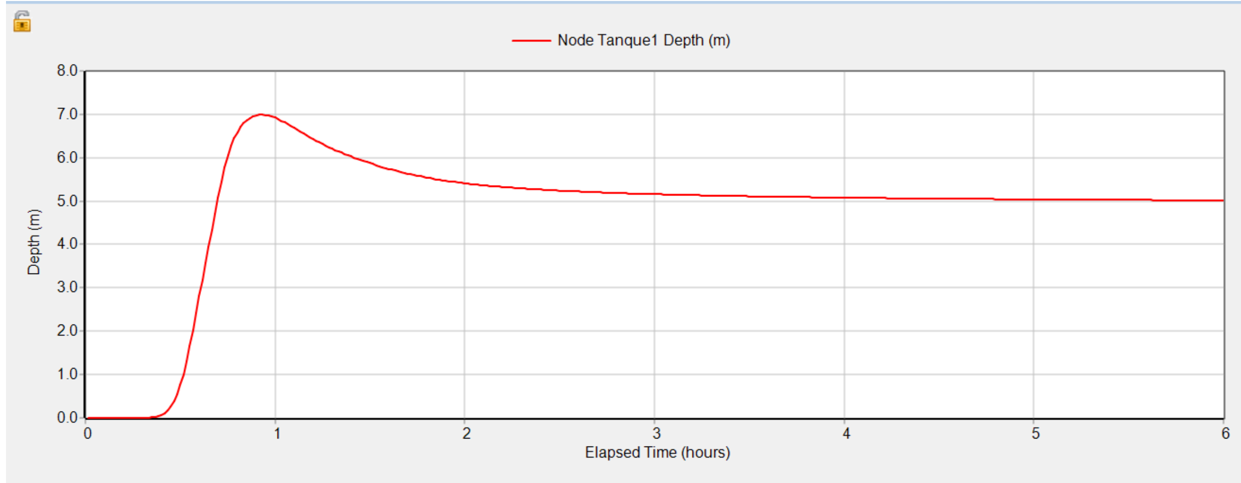


Figure 94. Inner depth of the storm tank long-term design storm. Alternative 2 without optimisation

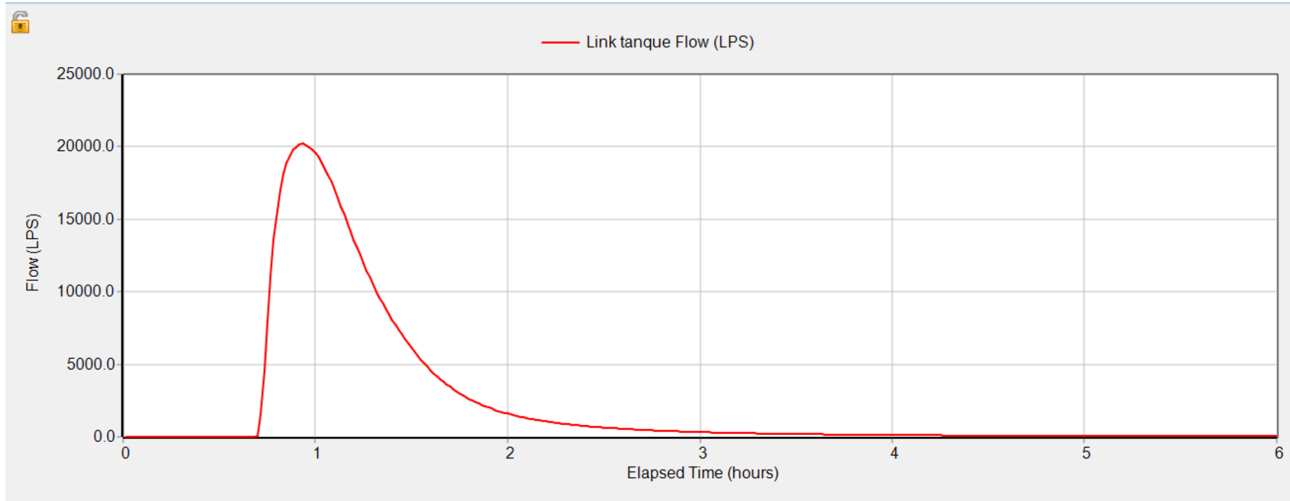


Figure 95. Discharged flow on long-term design storm. Alternative 2 without optimisation

As expected, it can be seen that the storm tank has considerably reduced the peak outflow. However, it has not been able to reduce it to the flow values of the whole catchment in its natural state.

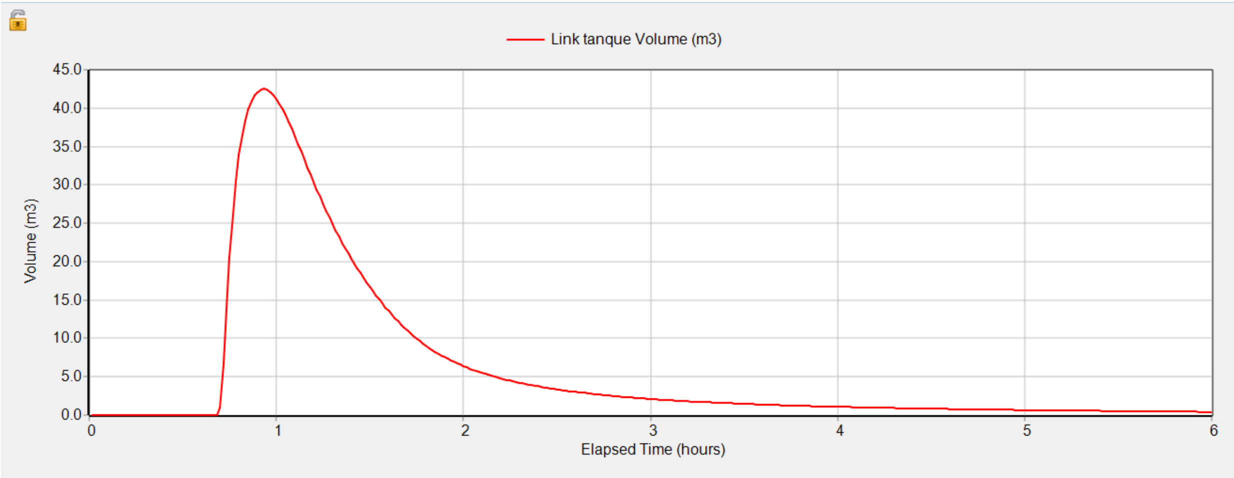


Figure 96. Discharged volume on long-term design storm. Alternative 2 without optimisation

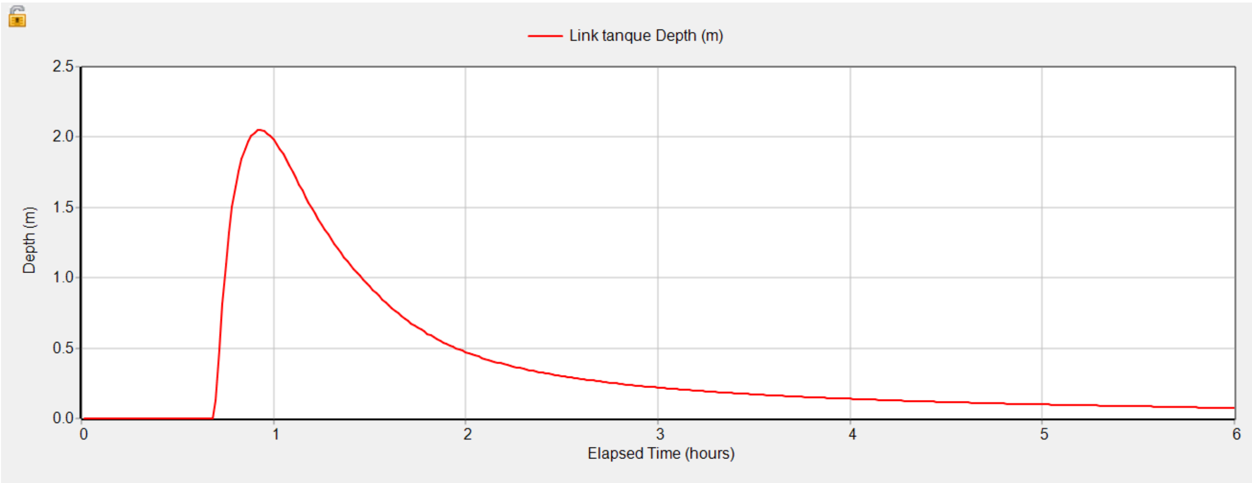


Figure 97. Depth on long-term design storm. Alternative 2 without optimisation

To achieve this objective, the design of the storm tank has been optimised to laminate the flow and reproduce in the outflow the flow in the natural state for the most unfavourable event, which is the present long-term design storm, and to achieve an outflow much lower for the short- and medium-term design storm, which, as will be seen below, is zero.

$$S_{tank} = \mathbf{6.220\ m2}$$
$$h_{tank} = \mathbf{10,0\ m}$$

$$V_{tank} = \mathbf{62.261\ m}$$

Table 42. Results on discharge point. Alternative 2 optimized

Design storm	Peak flow (m3/s)	Volume	Maximum depth
	m3/s	m3	m
Short-term	0,00	0	0,00
Medium-term	0,00	0	0,00
Long-term	6,88	18.222	1,01

5.7.2.4 Short-term design storm. Alternative 2 with optimisation

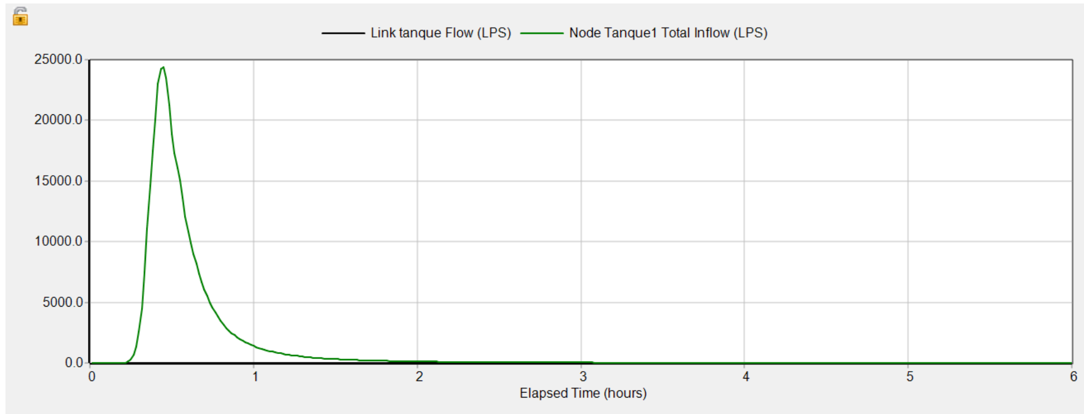


Figure 98. Comparison of inflow and outflow of the storm tank on short-term design storm. Alternative 2 with optimisation

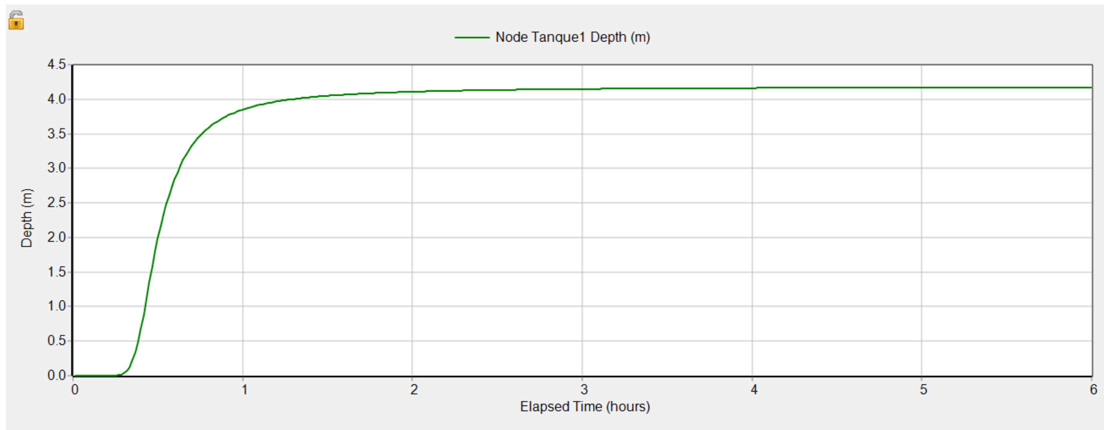


Figure 99. Inner depth of the storm tank short-term design storm. Alternative 2 with optimisation

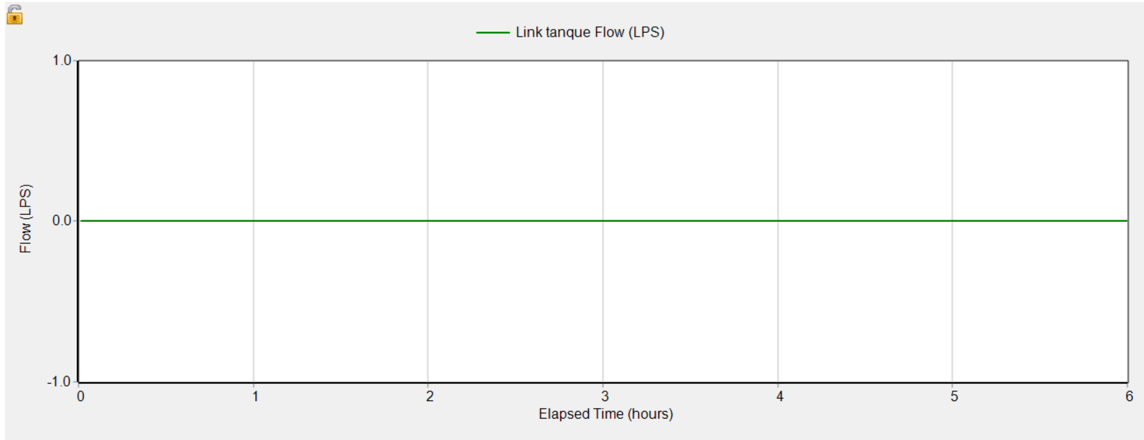


Figure 100. Discharged flow on short-term design storm. Alternative 2 with optimisation

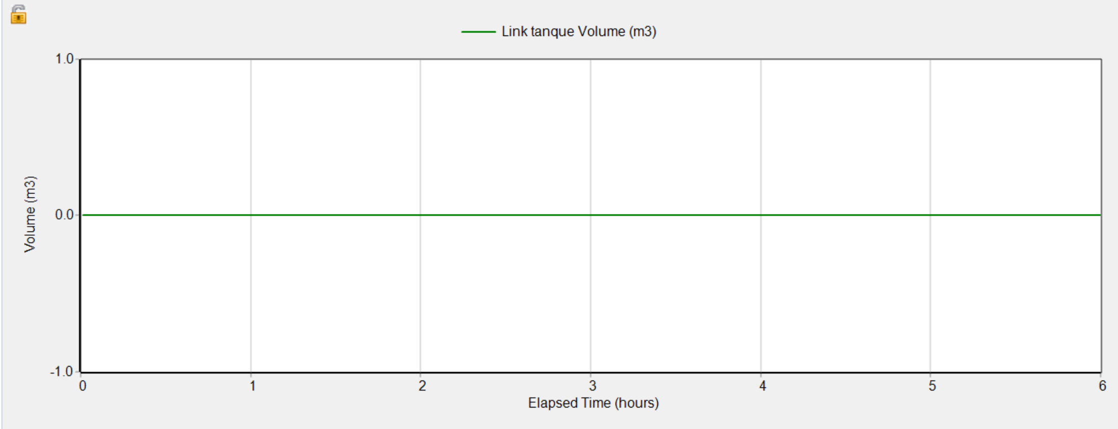


Figure 101. Discharged volume on short-term design storm. Alternative 2 with optimisation

5.7.2.5 Medium-term design storm. Alternative 2 with optimisation

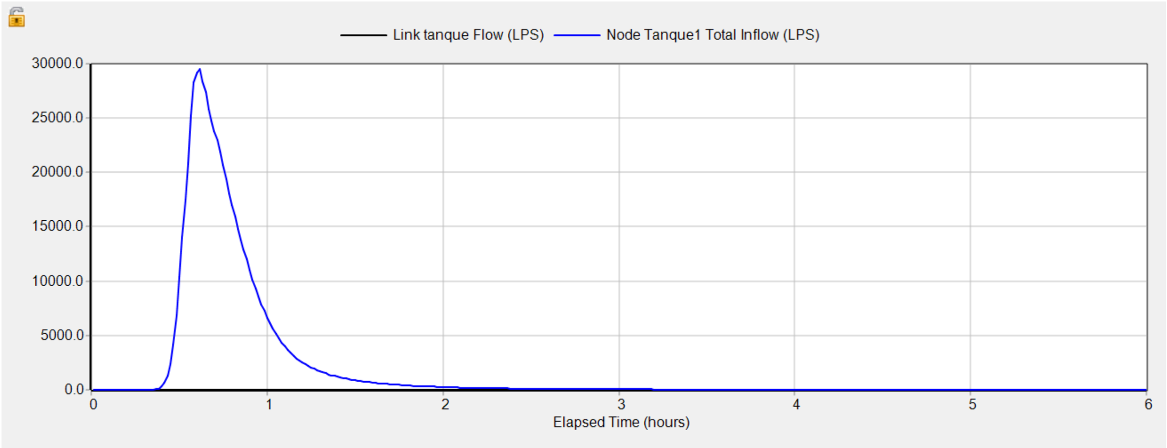


Figure 102. Comparison of inflow and outflow of the storm tank on medium-term design storm. Alternative 2 with optimisation

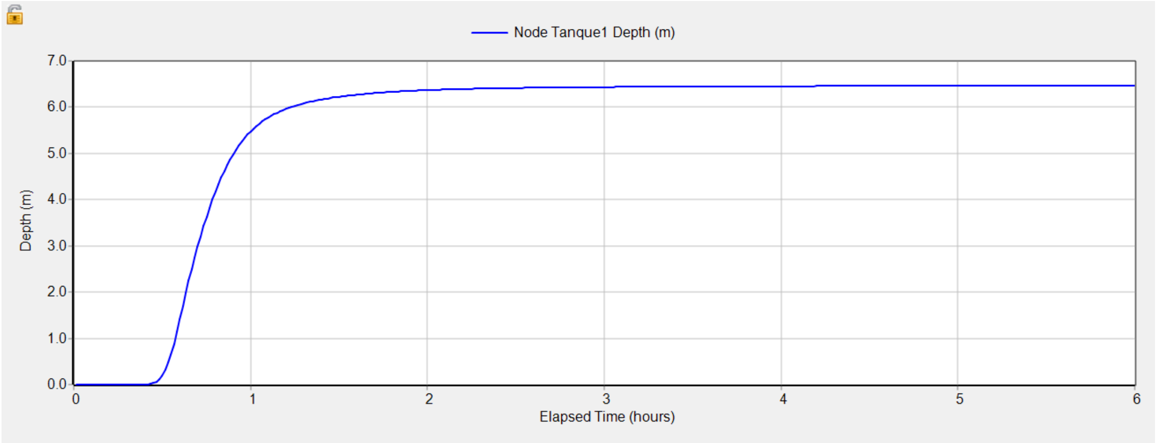


Figure 103. . Inner depth of the storm tank medium-term design storm. Alternative 2 with optimisation

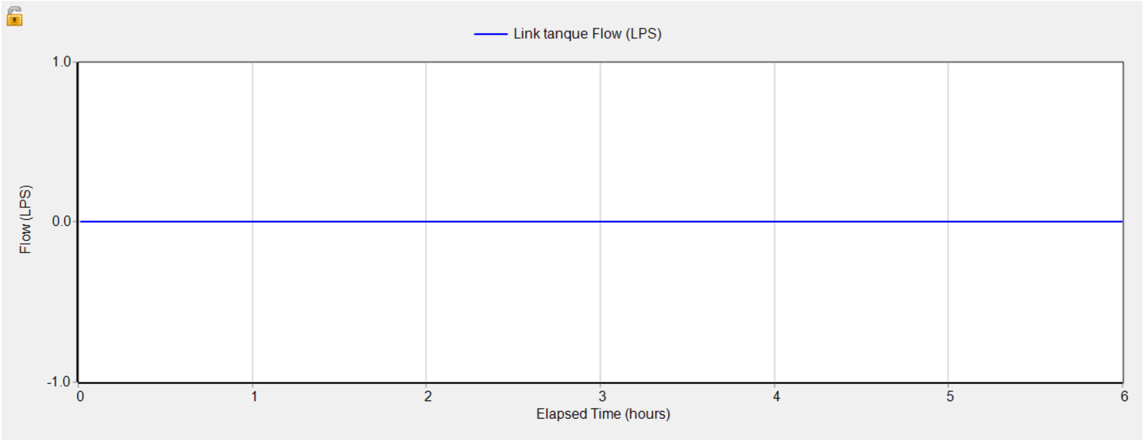


Figure 104. Discharged flow on medium-term design storm. Alternative 2 with optimisation

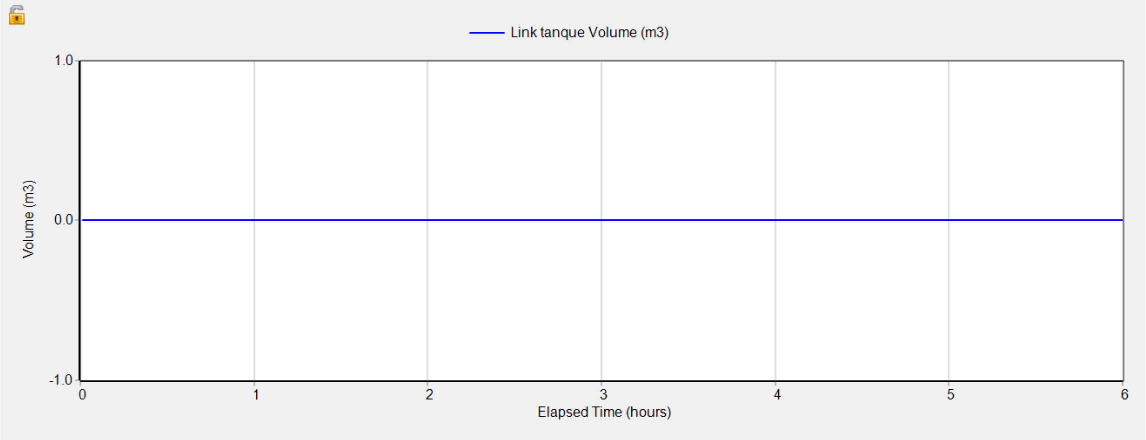


Figure 105. Discharged volume on medium-term design storm. Alternative 2 with optimisation

5.7.2.6 Long-term design storm. Alternative 2 with optimisation

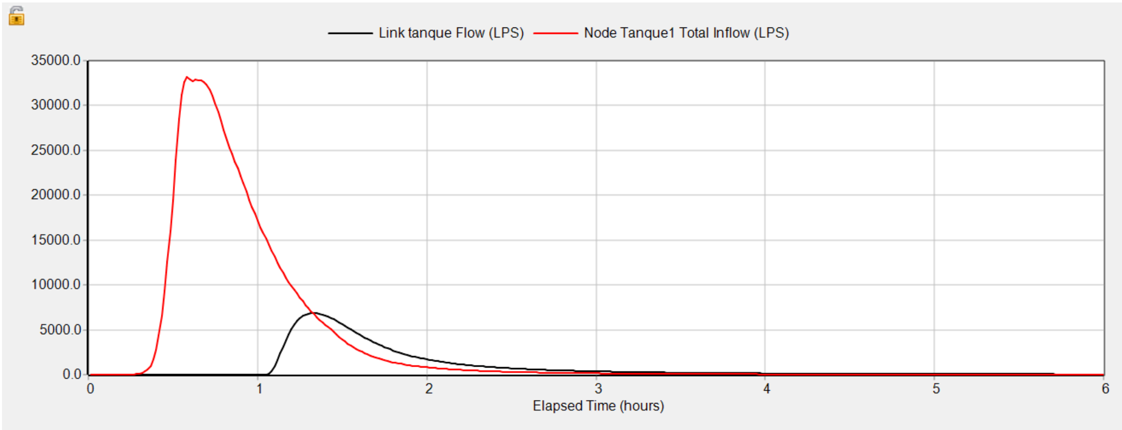


Figure 106. Comparison of inflow and outflow of the storm tank on long-term design storm. Alternative 2 with optimisation

The design has been made so that the outlet flow at the discharge point is equal to 6.88 l/s. As can be seen in the figure above.

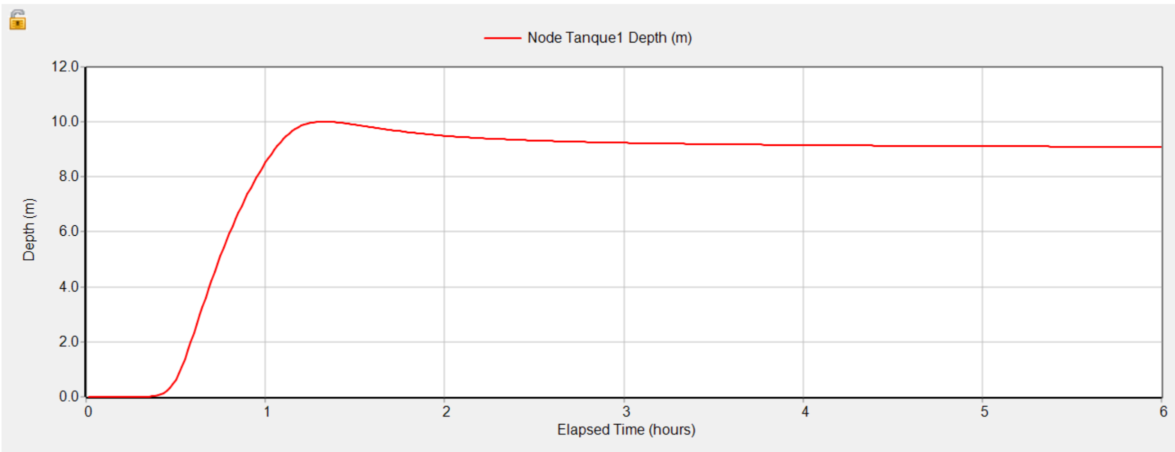


Figure 107. Inner depth of the storm tank long-term design storm. Alternative 2 with optimisatio

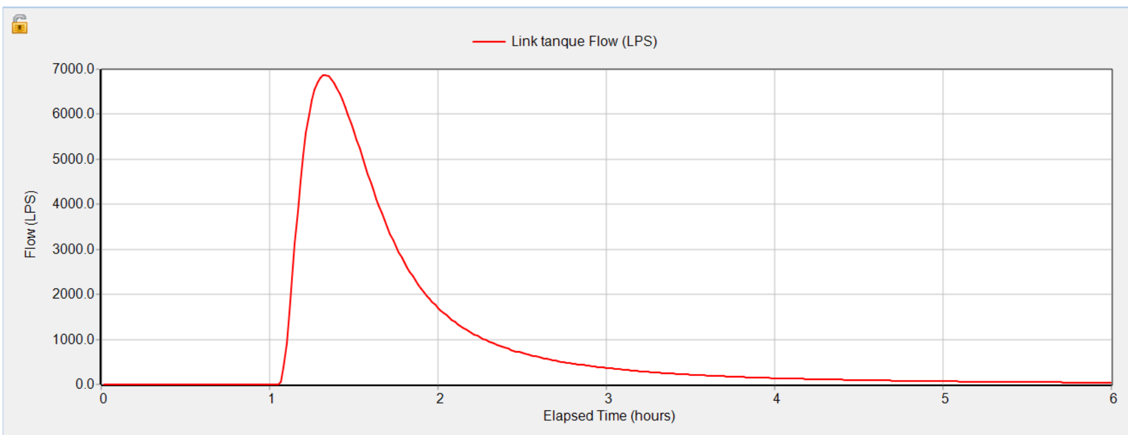


Figure 108. Discharged flow on long-term design storm. Alternative 2 with optimisation

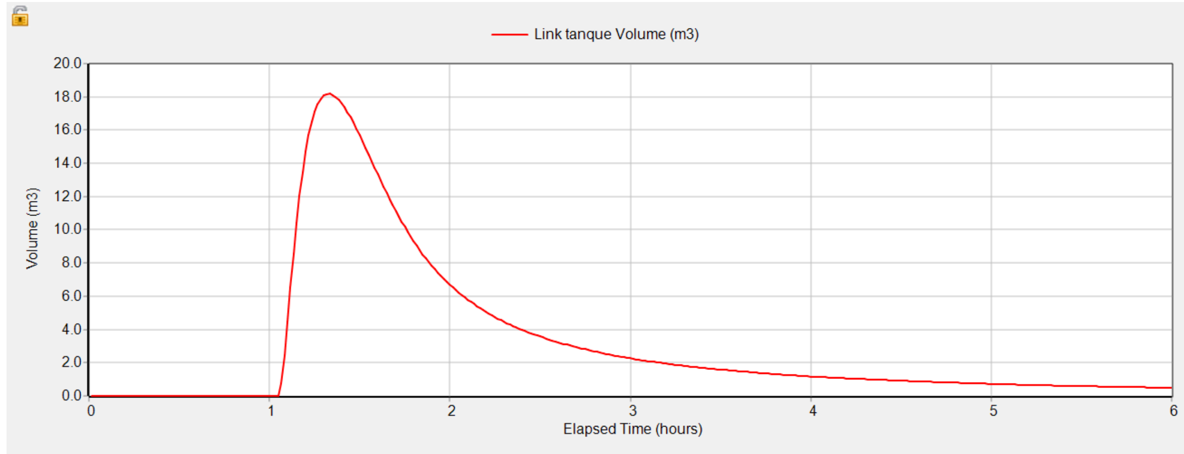


Figure 109. Discharged volume on long-term design storm. Alternative 2 with optimisation

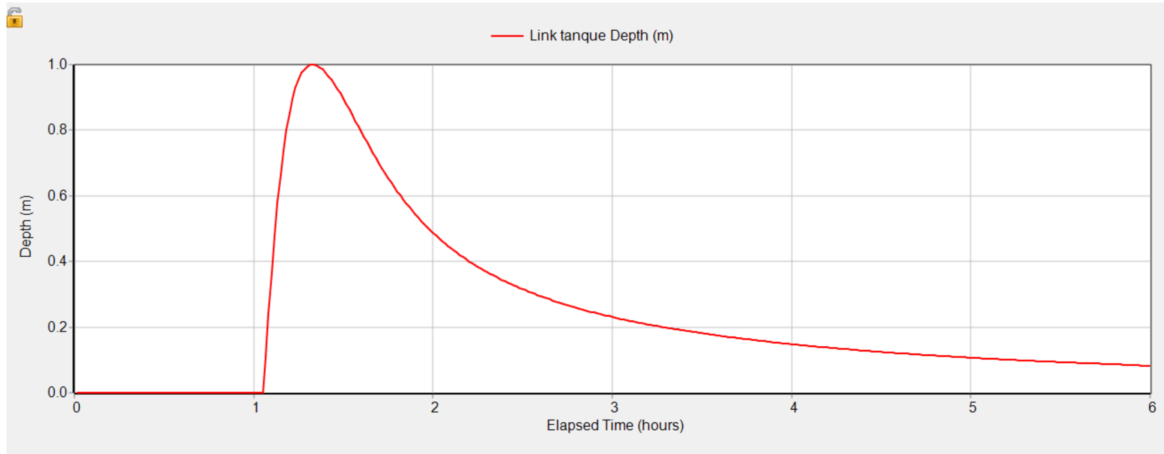


Figure 110. Depth on long-term design storm. Alternative 2 with optimisation

To conclude with this alternative, the optimised case reduces the outflow considerably. In the following table we can compare the flow reductions with the implementation of the storm tank with and without optimisation, marking in green those cases where the objective of achieving the natural flow as outflow is met.

Table 43. Reduction in peak outflow. Alternative 2

Design storm	Without optimisation	With optimisation
Short-term	100%	100%
Medium-term	76,8%	100%
Long-term	39,1%	79,4%

5.8 Modelling of alternative 3. Sustainable drainage system

For the modelling, the subcatchments with integrated sustainable drainage systems have been modified in the model.

In principle, the characteristics of the collector system network of modelling 1 have been kept, even knowing that the flow will be lower and that they would not be operating at their optimum capacity. The reason why this is done in this way is to be able to leave the possibility of implementing this solution in different phases and that in all of them the correct management of the drainage system is achieved.

In the section on the introduction to the SWMM program, the different types of SUDS that the program could simulate were discussed. In this section we are going to talk about the elements that have allowed rain gardens, infiltration basins and permeable pavements to be reproduced in the program and the characteristics that have been chosen for this purpose.

5.8.1 Sustainable drainage systems on SWMM

5.8.1.1 Pervious pavement

The modelling of pervious pavements has been chosen using the "pervious pavement" element of SWMM.

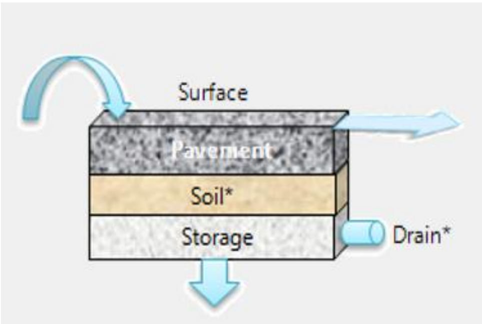


Figure 111. Pervious pavement scheme on SWMM

This configuration has three layers, the soil layer being optional, as well as the possibility of adding drainage and infiltration. The layers are:

- Pavement
- Soil
- Storage

All pervious pavements have the same characteristics, varying in surface area. The parameters introduced to model these pervious pavements are:

Table 44. Pervious pavement properties

Concept	Value	Unit	Definition
SURFACE LAYER			
Berm Height	0,0	mm	Maximum depth to which water can pond above the surface of the unit before overflow occurs
Vegetation Volume Fraction	0,0	-	The fraction of the volume within the storage depth filled with vegetation
Surface Roughness	0,02	-	Manning's n for overland flow over surface soil cover, pavement, roof surface or vegetative swale
Surface Slope	0,5	%	Slope of a roof surface, pavement surface or vegetative swale
PAVEMENT LAYER			
Thickness	200	mm	The thickness of the pavement layer
Void Ratio	0,25	-	The volume of void space relative to the volume of solids in the pavement for continuous systems or for the fill material used in modular systems
Impervious Surface	0	-	Ratio of impervious paver material to total area for modular systems
Permeability	250	mm/h	Permeability of the concrete or asphalt used in continuous systems or hydraulic conductivity of the fill material used in modular systems
Clogging Factor	0,0	-	Number of pavement layer void volumes of runoff treated it takes to completely clog the pavement
Regeneration Interval	0,0	days	
Regeneration Fraction	0,0	-	
STORAGE LAYER			
Thickness	450	mm	Thickness of a gravel layer
Void Ratio	0,5	-	The volume of void space relative to the volume of solids in the layer
Seepage Rate	0,0	-	The rate at which water seeps into the native soil below the layer
Clogging Factor	0,0	-	Total volume of treated runoff it takes to completely clog the bottom of the layer divided by the void volume of the layer
DRAIN			
Flow coefficient	2,12	-	Coefficient C used on equation of flow rate : $q = C * h^n$
Flow Exponent	0,5	-	Exponent n used on equation of flow rate : $q = C * h^n$
Offset	5,0	mm	The height of the drain line above the bottom of a storage laye

5.8.1.2 Rain gardens

SWMM offers an option called rain gardens, but this option does not have the storage layer, which is key for the simulation of the case study. Therefore, the rain gardens have been modelled with the bio-retention cell element, which has exactly the same parameters, but also has the storage layer and drain option, which is necessary when rain gardens are at the head of the SUDS chain.

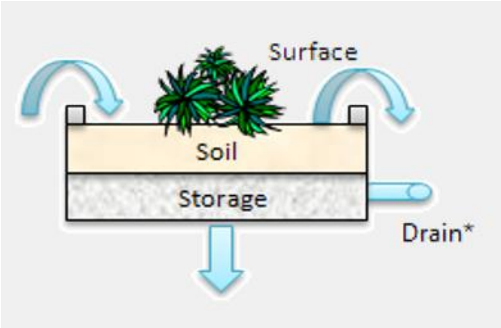


Figure 112. Bio-retention cell scheme on SWMM

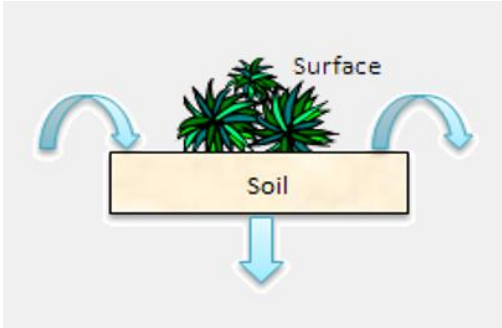


Figure 113. Rain garden scheme on SWMM

In the case study there are two different SUDS sections. One section that allows infiltration and has no drain (JLL_inf) and another section that does not allow infiltration and has a drain (JLLL_dren).

The characteristics of both are presented in the following tables:

Table 45. Rain garden with drain (JLL_inf) properties

Concept	Value	Unit	Definition
SURFACE LAYER			
Berm Height	300	mm	Maximum depth to which water can pond above the surface of the unit before overflow occurs
Vegetation Volume Fraction	0,1	-	The fraction of the volume within the storage depth filled with vegetation
Surface Roughness	0,2	-	Manning's n for overland flow over surface soil cover, pavement, roof surface or vegetative swale
Surface Slope	0,5	%	Slope of a roof surface, pavement surface or vegetative swale
SOIL LAYER			
Thickness	300	mm	The thickness of the soil layer
Porosity	0,5	-	he volume of pore space relative to total volume of soi
Field Capacity	0,06	-	Volume of pore water relative to total volume after the soil has been allowed to drain fully
Wilting Point	0,02	-	Volume of pore water relative to total volume for a well dried soil where only bound water remains
Conductivity	250	mm/h	Hydraulic conductivity for the fully saturated soil
Conductivity Slope	45	-	Slope of the curve of log(conductivity) versus soil moisture content (dimensionless)
Suction Head	3,5	mm	The average value of soil capillary suction along the wetting front
STORAGE LAYER			
Thickness	450	mm	Thickness of a gravel layer
Void Ratio	0,5	-	The volume of void space relative to the volume of solids in the layer
Seepage Rate	12,9	mm/h	The rate at which water seeps into the native soil below the layer
Clogging Factor	0,0	-	Total volume of treated runoff it takes to completely clog the bottom of the layer divided by the void volume of the layer
DRAIN			
Flow coefficient	0,0	-	Coefficient C used on equation of flow rate : $q = C * h^n$
Flow Exponent	0,0	-	Exponent n used on equation of flow rate : $q = C * h^n$
Offset	0,0	mm	The height of the drain line above the bottom of a storage layeR

Table 46. Rain garden with drain (JLL_dren) properties

Concept	Value	Unit	Definition
SURFACE LAYER			
Berm Height	300	mm	Maximum depth to which water can pond above the surface of the unit before overflow occurs
Vegetation Volume Fraction	0,1	-	The fraction of the volume within the storage depth filled with vegetation
Surface Roughness	0,2	-	Manning's n for overland flow over surface soil cover, pavement, roof surface or vegetative swale
Surface Slope	0,5	%	Slope of a roof surface, pavement surface or vegetative swale
SOIL LAYER			
Thickness	300	mm	The thickness of the soil layer
Porosity	0,5	-	he volume of pore space relative to total volume of soi
Field Capacity	0,06	-	Volume of pore water relative to total volume after the soil has been allowed to drain fully
Wilting Point	0,02	-	Volume of pore water relative to total volume for a well dried soil where only bound water remains
Conductivity	250	mm/h	Hydraulic conductivity for the fully saturated soil
Conductivity Slope	45	-	Slope of the curve of log(conductivity) versus soil moisture content (dimensionless)
Suction Head	3,5	mm	The average value of soil capillary suction along the wetting front
STORAGE LAYER			
Thickness	450	mm	Thickness of a gravel layer
Void Ratio	0,5	-	The volume of void space relative to the volume of solids in the layer
Seepage Rate	0,0	mm/h	The rate at which water seeps into the native soil below the layer
Clogging Factor	0,0	-	Total volume of treated runoff it takes to completely clog the bottom of the layer divided by the void volume of the layer
DRAIN			
Flow coefficient	2,28	-	Coefficient C used on equation of flow rate : $q = C * h^n$
Flow Exponent	0,5	-	Exponent n used on equation of flow rate : $q = C * h^n$
Offset	5,0	mm	The height of the drain line above the bottom of a storage layeR

5.8.1.3 Infiltration basin

The infiltration basins have been modelled with the bio-retention cell element. Its cross-section is similar to the rain garden that allows infiltration, but with a higher free water height.

Table 47. Infiltration basin properties

Concept	Value	Unit	Definition
SURFACE LAYER			
Berm Height	600	mm	Maximum depth to which water can pond above the surface of the unit before overflow occurs
Vegetation Volume Fraction	0,1	-	The fraction of the volume within the storage depth filled with vegetation
Surface Roughness	0,2	-	Manning's n for overland flow over surface soil cover, pavement, roof surface or vegetative swale
Surface Slope	0,5	%	Slope of a roof surface, pavement surface or vegetative swale
SOIL LAYER			
Thickness	300	mm	The thickness of the soil layer
Porosity	0,5	-	he volume of pore space relative to total volume of soi
Field Capacity	0,06	-	Volume of pore water relative to total volume after the soil has been allowed to drain fully
Wilting Point	0,02	-	Volume of pore water relative to total volume for a well dried soil where only bound water remains
Conductivity	250	mm/h	Hydraulic conductivity for the fully saturated soil
Conductivity Slope	45	-	Slope of the curve of log(conductivity) versus soil moisture content (dimensionless)
Suction Head	3,5	mm	The average value of soil capillary suction along the wetting front
STORAGE LAYER			
Thickness	450	mm	Thickness of a gravel layer
Void Ratio	0,5	-	The volume of void space relative to the volume of solids in the layer
Seepage Rate	12,9	mm/h	The rate at which water seeps into the native soil below the layer
Clogging Factor	0,0	-	Total volume of treated runoff it takes to completely clog the bottom of the layer divided by the void volume of the layer
DRAIN			
Flow coefficient	0,0	-	Coefficient C used on equation of flow rate : $q = C * h^n$
Flow Exponent	0,0	-	Exponent n used on equation of flow rate : $q = C * h^n$
Offset	0,0	mm	The height of the drain line above the bottom of a storage layer

5.8.2 Modelling representation alternative 3

The modelling representation of scenario 3 keeps the drainage system conduits of modelling 1, but the catchments have been modified to add the SuDS in the subcatchments. Thus, the areas of the subcatchments representing the subcatchments have been reduced and another subcatchment has been added to represent the SuDS with one hundred percent of the area occupied by the SuDS.

The N1 and N2 catchments have a similar connection logic system. The N3 catchment is a little more complex.

First, we will discuss how the SuDS in the N1 and N2 catchments have been modelled. A chain of connections has been made in which the runoff from the parcel subcatchment passes to the pervious pavement subcatchment, then the runoff collected by the pervious pavements is conveyed by drains to the main drainage system. In this way we have ensured that all runoff from the street subcatchments has passed through the pervious pavements.

In contrast to modelling 1, the drainage system is split so that just before reaching the end of the catchment, it discharges into the infiltration basin located in the green zone. Anything that exceeds the capacity limit of the SuDS will be discharged into the continuation of the drainage system to the next catchment, at manhole P121 and P226 for catchments N1 and N2, respectively. This configuration allows all water from the street subcatchments to be treated by the previously proposed SuDS chain to meet pollutant mitigation requirements. This is also because, although the action on the private subcatchments is not the subject of the study, their runoff is a very large proportion of the total runoff and to reduce the peak flow of the whole study area it is much more effective if this runoff can pass through at least one SuDS element.

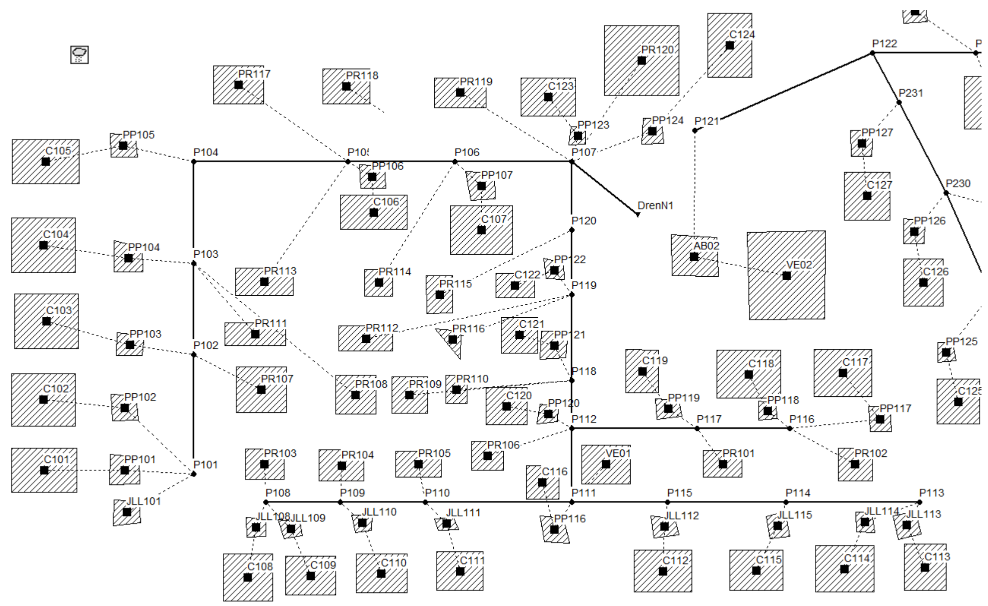


Figure 114. Modelling catchment N1. Alternative 3

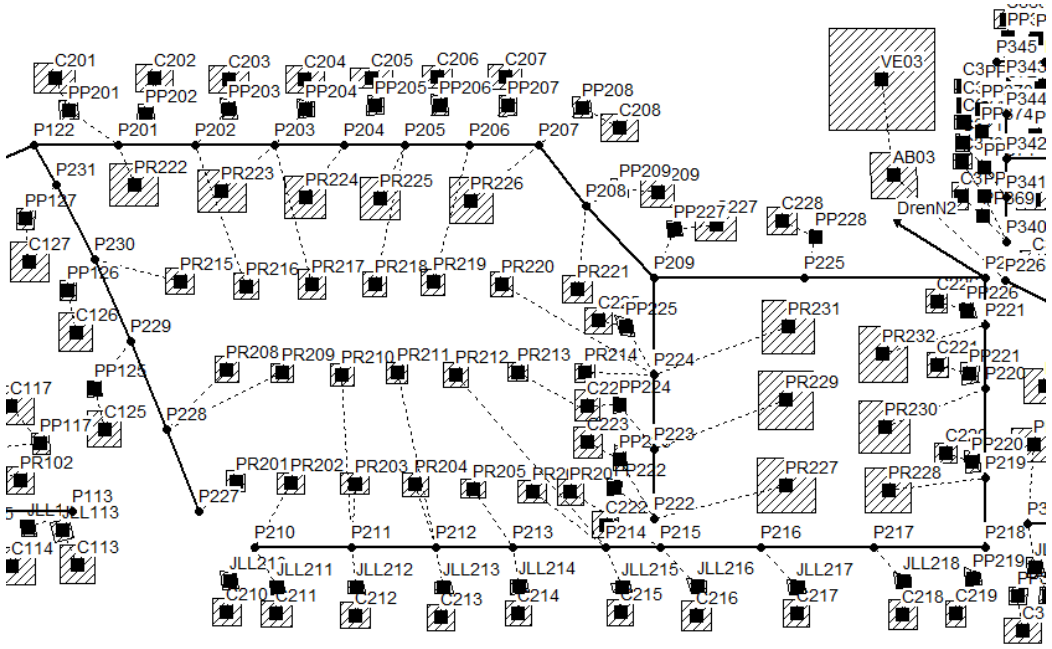


Figure 115. Modelling catchment N2. Alternative 3

In the case of catchment N3, the distribution is different because there is no large green area in which a SuDS element, of the infiltration area type, can be implemented, so the configuration of the SuDS chain has to be done subcatchment by catchment.

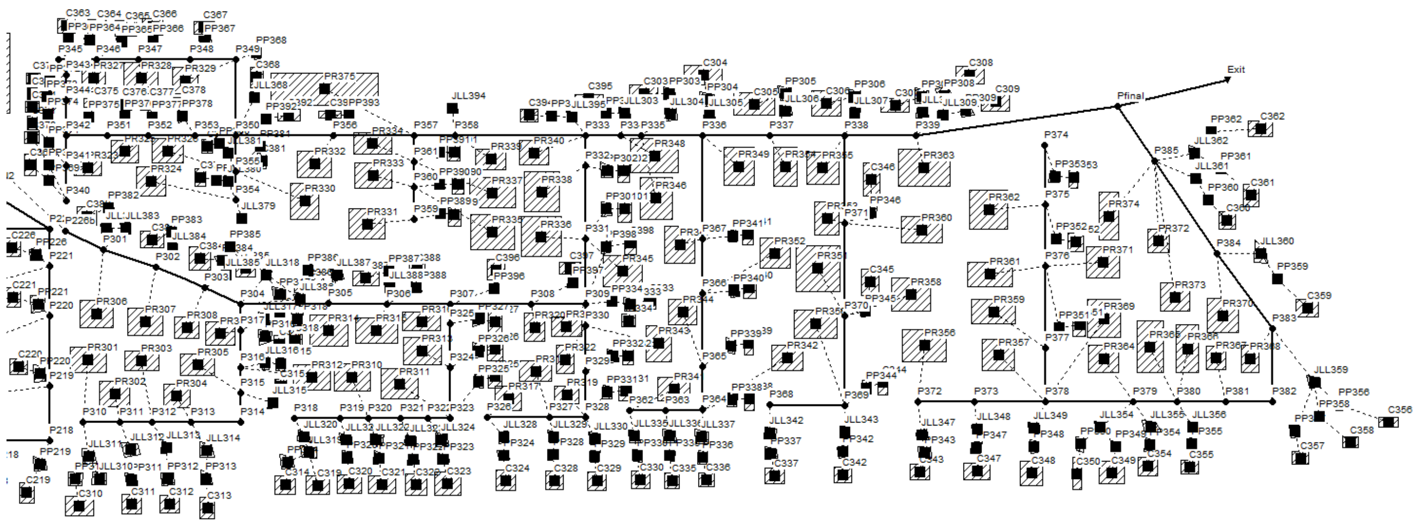


Figure 116. Modelling catchment N1. Alternative 3

In this way, as can be seen in the detailed example of the SUDS connection of segments 3F and 3Z with segment 3G, all runoff from the street subcatchment passes through a pervious pavement and then through a rain garden that allows infiltration, connecting the excess that exceeds the storage limit of the rain garden to the drainage system.

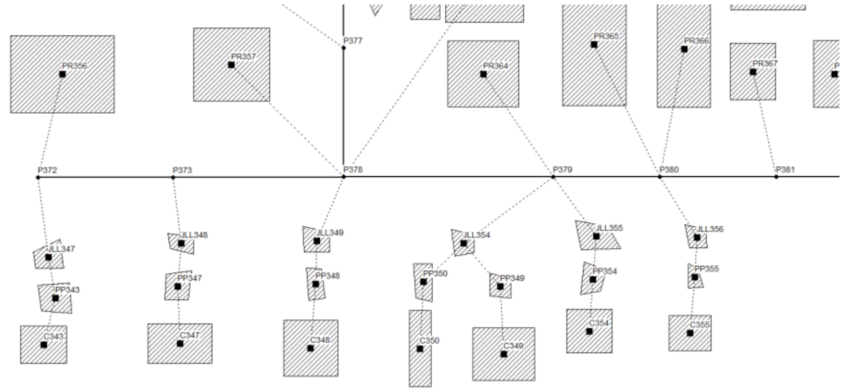


Figure 117. Detail of SUDS connection segments 3F, 2Z and 3G. Alternative 3

5.8.3 Modelling results alternative 3

After modelling alternative 3 for a short-, medium- and longterm design storm, the following values have been obtained at the discharge point:

Table 48. Results on discharge point. Alternative 3

Design storm	Peak flow (m3/s)	Volume	Maximum depth
	m3/s	m3	m
Short-term	12,69	13.101	1,60
Medium-term	14,14	20.667	1,69
Long-term	15,91	39.584	1,81

Achieving a considerable reduction in peak flow in the case of catchment N3 is much more complex than the rest. Because, firstly, it does not have an extensive green area in which to implement a SuDS such as an infiltration basin, a bioretention area or even a detention basin. These elements have a large storage capacity, and their implementation would help a lot to laminate the flow. Secondly, because a large part of the runoff comes from private subcatchments and, in the case of catchment N3, it is not possible for this runoff to pass through a SuDS element as was the case with catchment N1 and N2.

The consequence of this configuration is that, although the peak flow is considerably reduced, it has not been possible to achieve a discharge flow equal to or less than the flow in a natural state without acting on the private subcatchments.

Table 49. Discharge peak flow reduction. Alternative 3

Design storm	Peak flow reduction
Short-term	48,18%
Medium-term	52,41%

Long-term	52,21%

In the following, the graphs on the flow rate and volume discharged, as well as the depth in the final segment, will be presented. In addition, it has been considered interesting to present the flow that passes through the end of the N1 and N2 catchment to demonstrate how the SUDS elements are capable of retaining all or almost all of the water from the N1 and N2 catchments and that it is the N3 catchment that actually generates the runoff of the flow that is discharged into the receiving medium.

5.8.3.1 Short-term design storm. Alternative 3

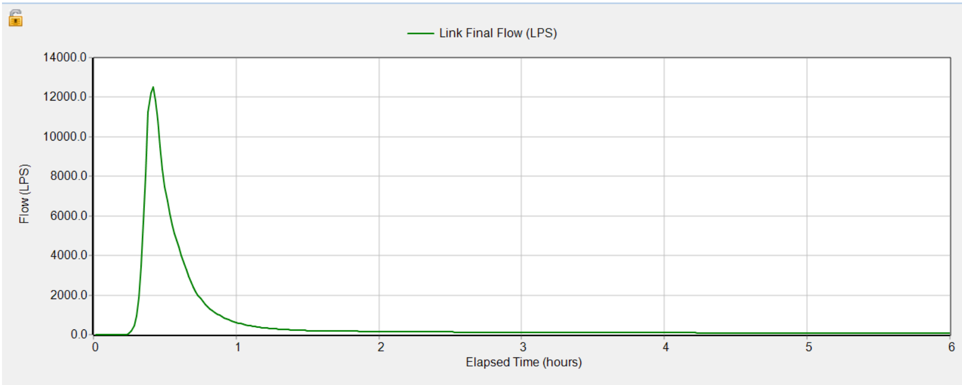


Figure 118. Discharged flow on short-term design storm. Alternative 3

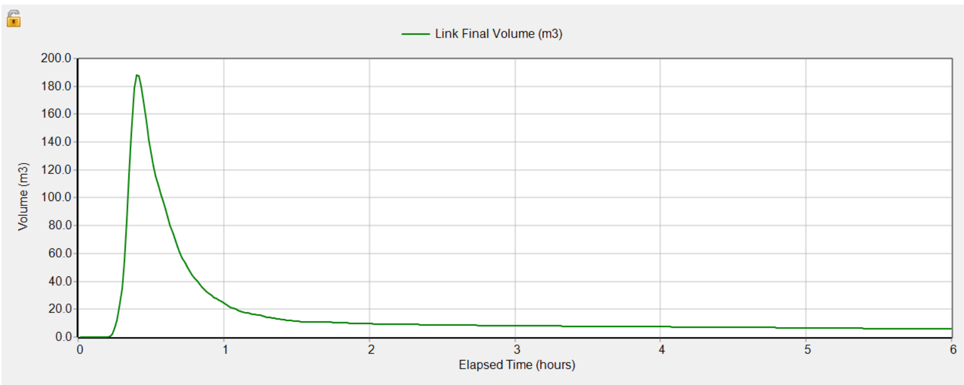


Figure 119. Discharged volume on short-term design storm. Alternative 3

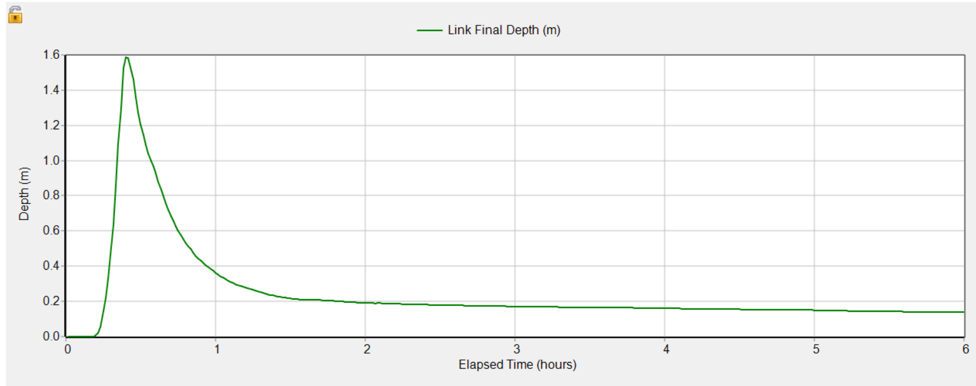


Figure 120. Depth on short-term design storm. Alternative 3

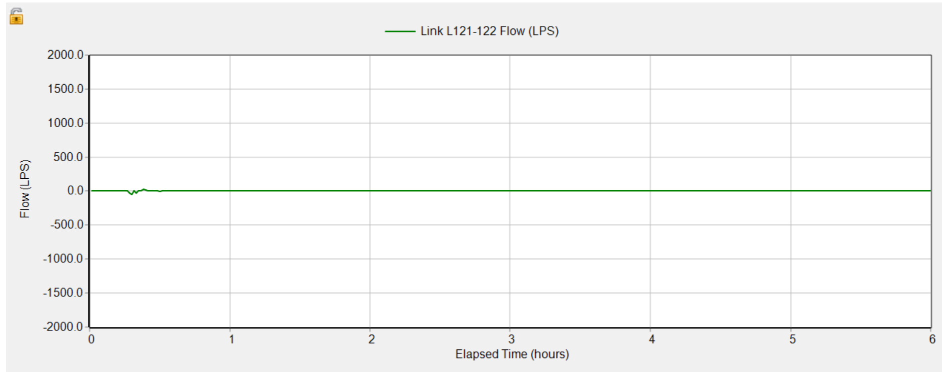


Figure 121. Final flow in catchment N1 on short-term design storm. Alternative 3

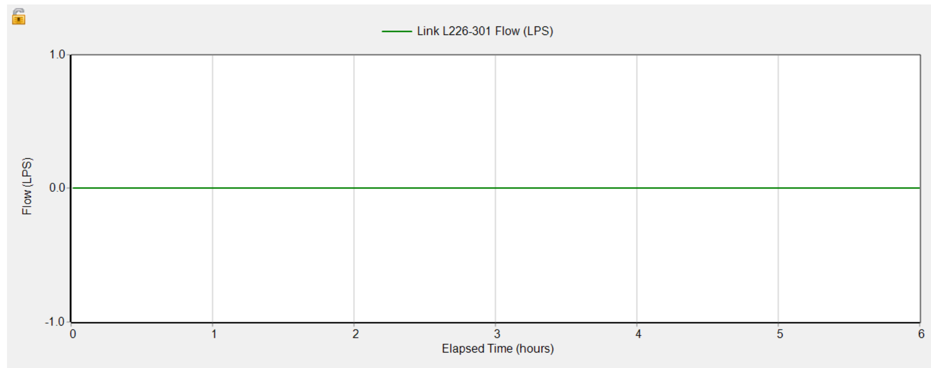


Figure 122. Final flow in catchment N2 on short-term design storm. Alternative 3

5.8.3.2 Medium-term design storm. Alternative 3

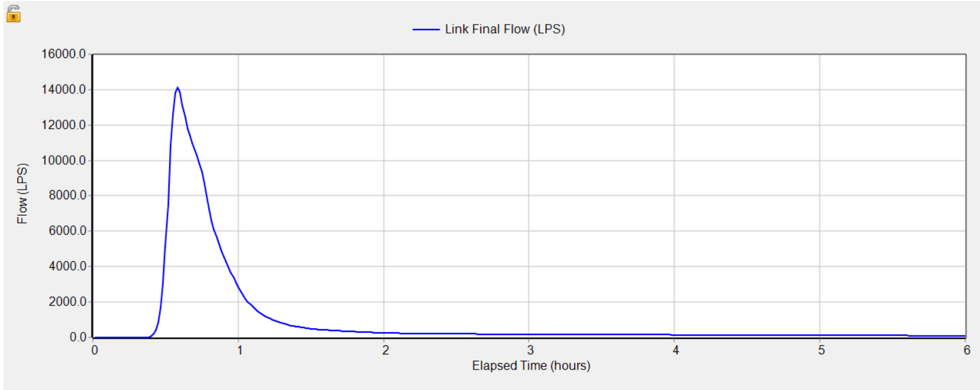


Figure 123. Discharged flow on medium-term design storm. Alternative 3

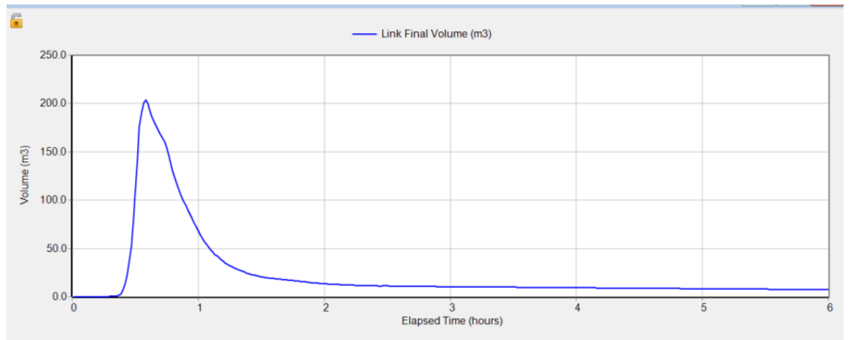


Figure 124. Discharged volume on medium-term design storm. Alternative 3

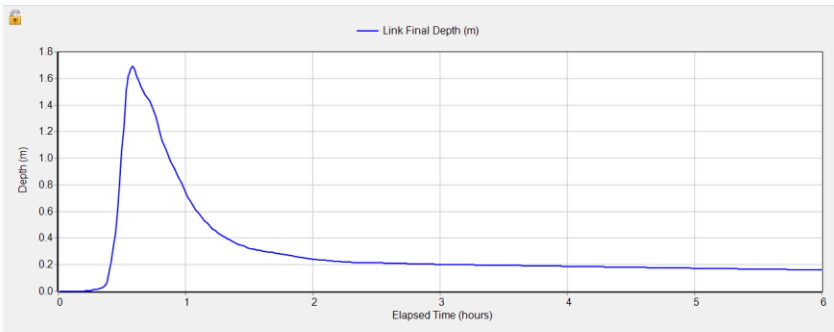


Figure 125. Depth on medium-term design storm. Alternative 3

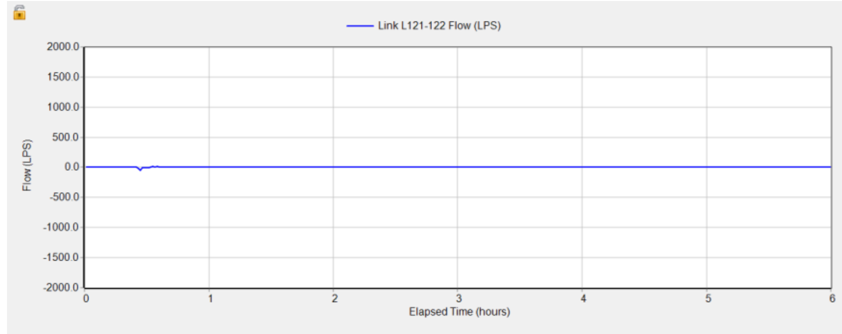


Figure 126. Final flow in catchment N1 on medium-term design storm. Alternative 3

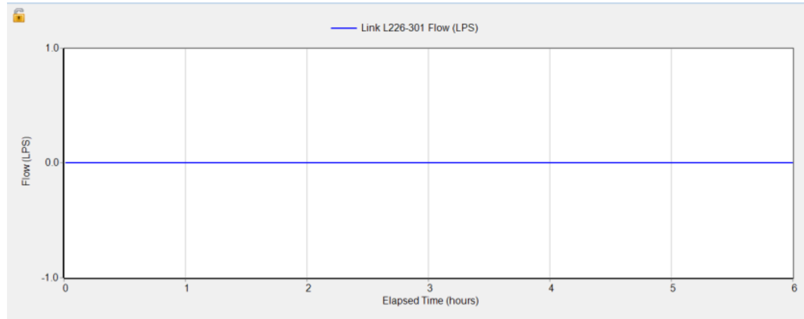


Figure 127. Final flow in catchment N2 on medium-term design storm. Alternative 3

5.8.3.3 Long-term design storm. Alternative 3

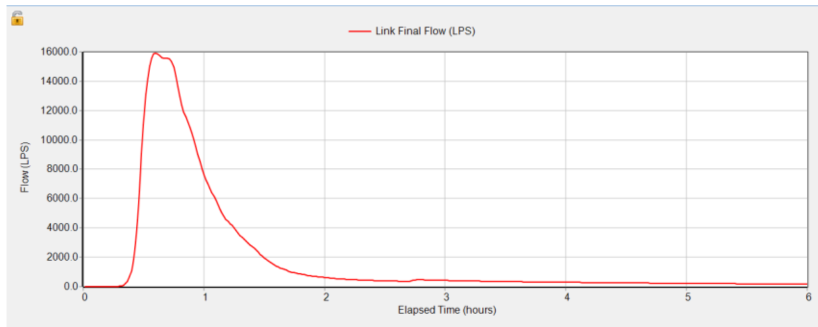


Figure 128. Discharged flow on long-term design storm. Alternative 3

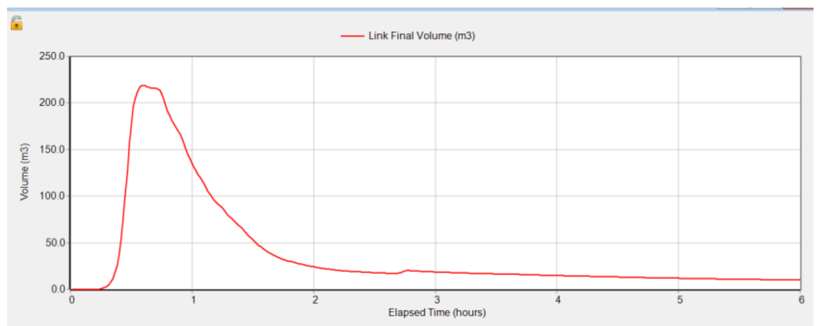


Figure 129. Discharged volume on long-term design storm. Alternative 3

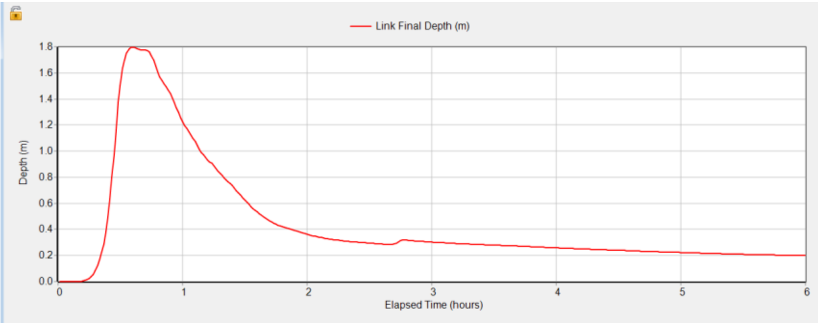


Figure 130. Depth on medium-term design storm. Alternative 3

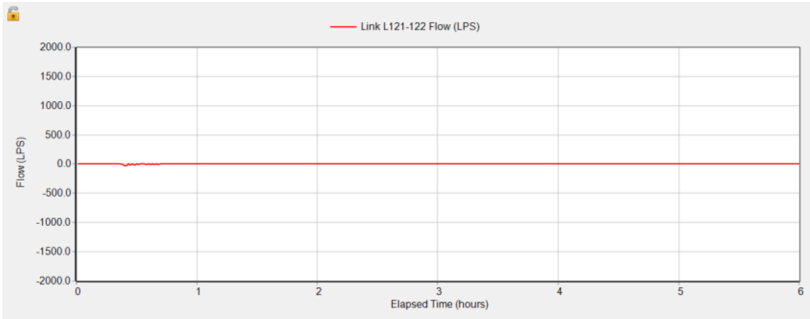


Figure 131. Final flow in catchment N1 on long-term design storm. Alternative 3

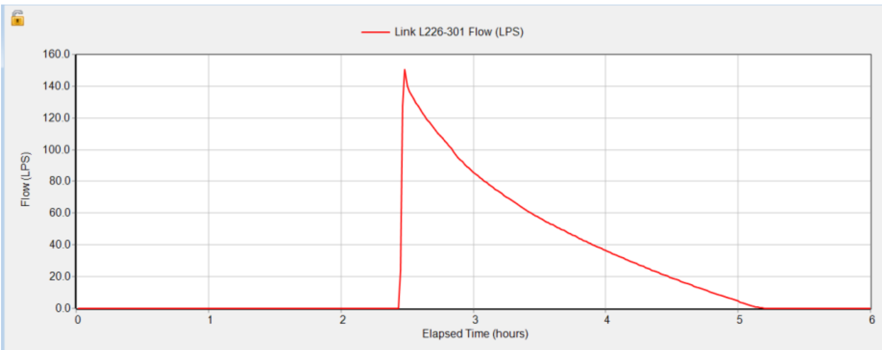


Figure 132. Final flow in catchment N2 on short-term design storm. Alternative 3

5.9 Modelling alternative 4. SuDS with stormwater tank

The last alternative proposes to use the advantages of alternative 2 and alternative 3 to generate a new scenario. In this alternative 4 the sustainable drainage systems are implemented and in addition a storm tank is added in series at the end of the collector system.

The storm tank replicates alternative 2 but with a smaller volume. This tank makes it possible to reduce the discharge flow and thus achieve the objective of discharging the flow in its natural state in the most disadvantageous storm, corresponding to the one with the longest duration.

5.9.1 Modelling representation alternative 4

The representation in the SWMM tool is based on modelling 3 and a storage unit is added in the final section with enough volume to be able to manage the runoff water so that the final discharge is the same as in the natural state.

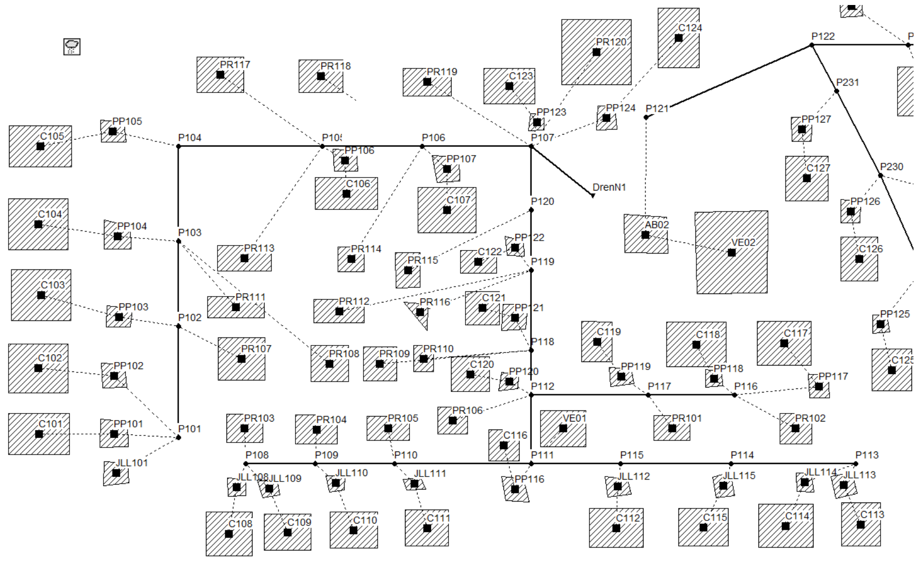


Figure 133. Modelling catchment N1. Alternative 4

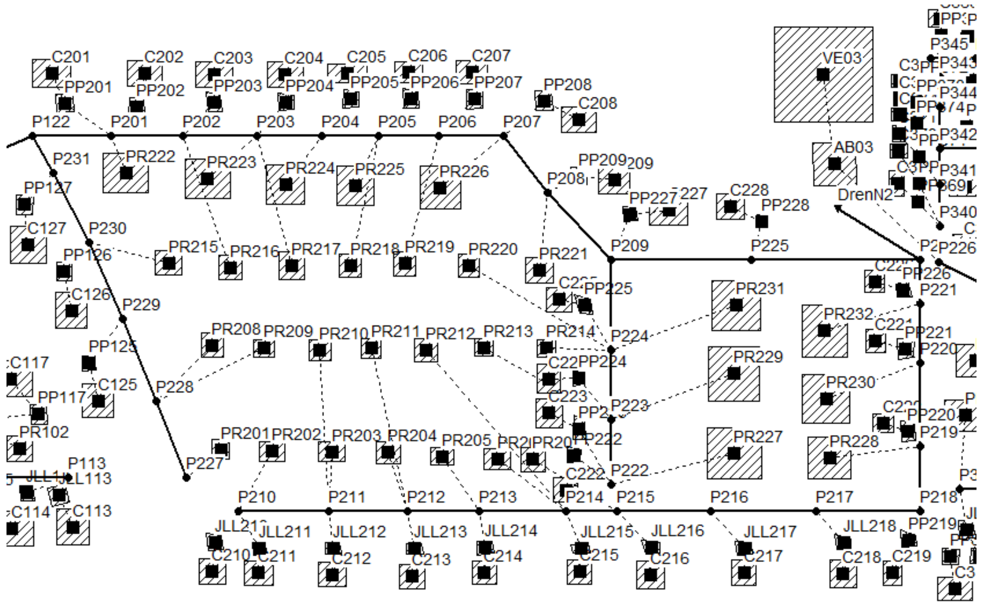


Figure 134. Modelling catchment N2. Alternative 4

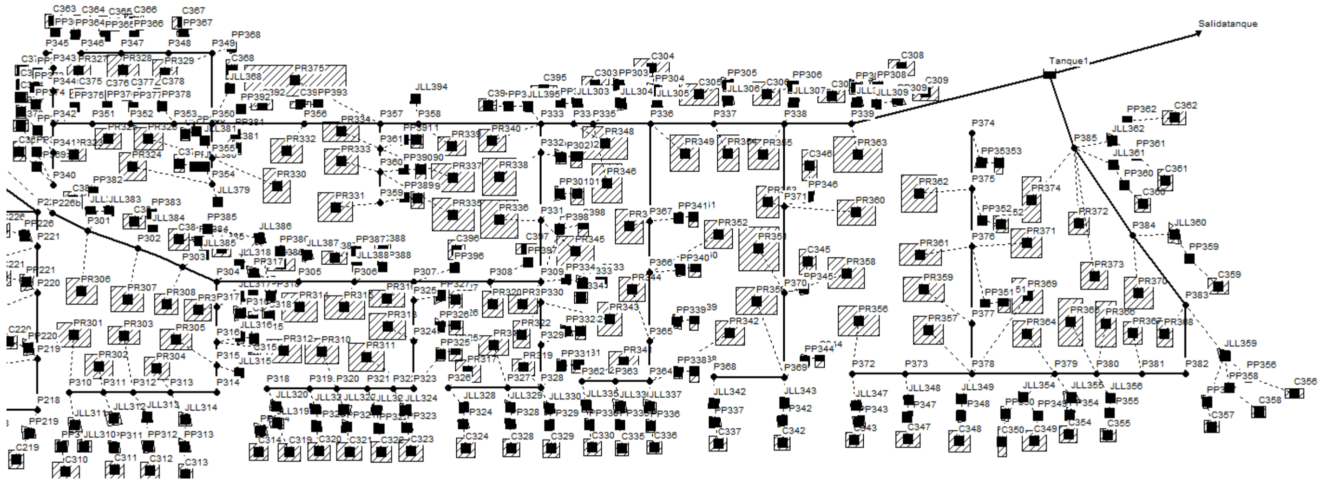


Figure 135. Modelling catchment N3. Alternative 4

5.9.2 Modelling results alternative 4

The parameters of the inlet flow to the storm tank are as follows:

Table 50. Inflow parameters to stormwater tank

Design storm	**Peak flow (m3/s)**	**Volume**
	m3/s	**m3**
Short-term	12,69	13.101
Medium-term	14,14	20.667
Long-term	15,91	39.584

Thus, after successive simulations adjusting the dimensions of the storm tank, we have arrived at the solution where the outlet flow is equal to the natural discharge flow, which is:

$$Q_{natural} = \mathbf{6,88\ m3/s}$$

The dimensions of the proposed storm tank are:

$$S_{tank} = \mathbf{3.655\ m2}$$
$$h_{tank} = \mathbf{6,4\ m}$$

$$V_{tank} = \mathbf{23.396\ m}$$

Table 51. Results on discharge point. Alternative 4

Design storm	**Peak flow (m3/s)**	**Volume**	**Maximum depth**
	m3/s	**m3**	**m**
Short-term	0,00	0	0,00
Medium-term	0,00	0	0,00
Long-term	6,88	17,868	1,01

5.9.2.1 Short-term design storm. Alternative 4

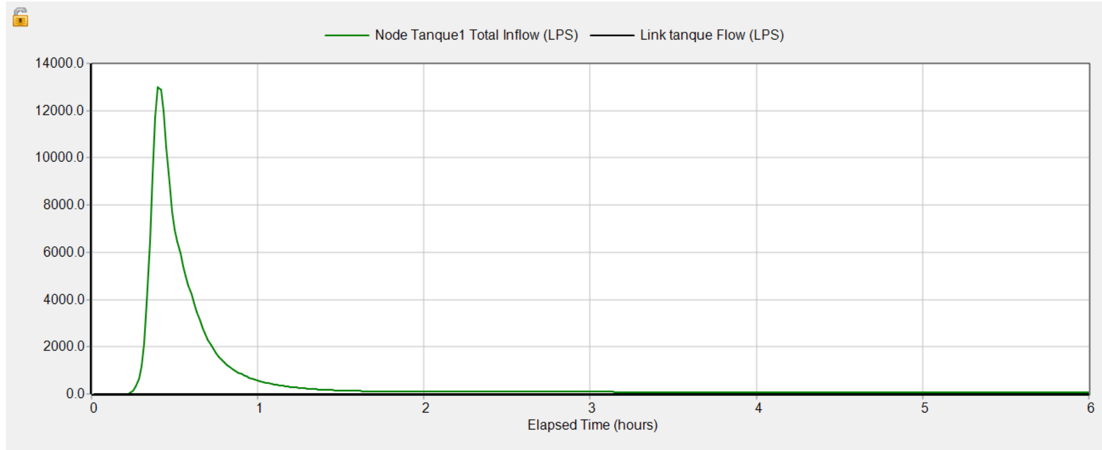


Figure 136. Comparison of inflow and outflow of the storm tank on short-term design storm. Alternative 4

In the short-term design storm of alternative 4, the design of the storm tank is oversized so that nothing is discharged into the environment. This is because the storm tank has been designed to meet the objectives in the most unfavourable design storm, which is the long-term design storm.

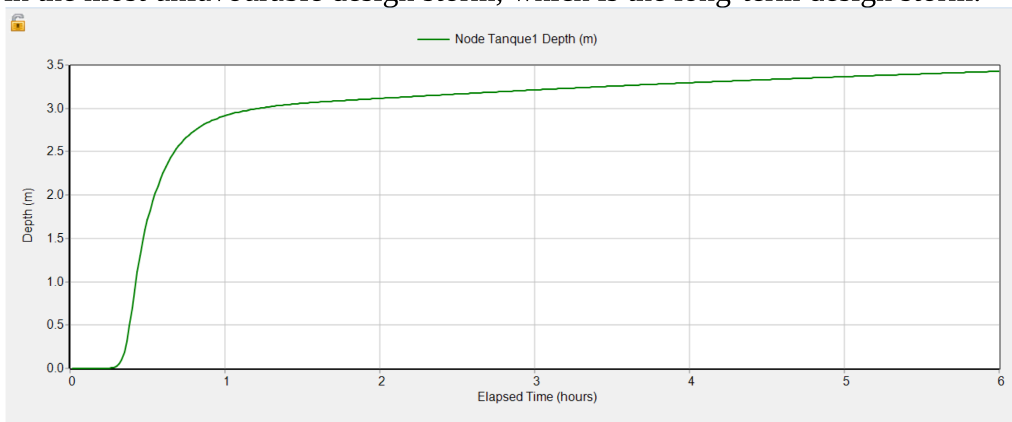


Figure 137. Inner depth of the storm tank short-term design storm. Alternative 4

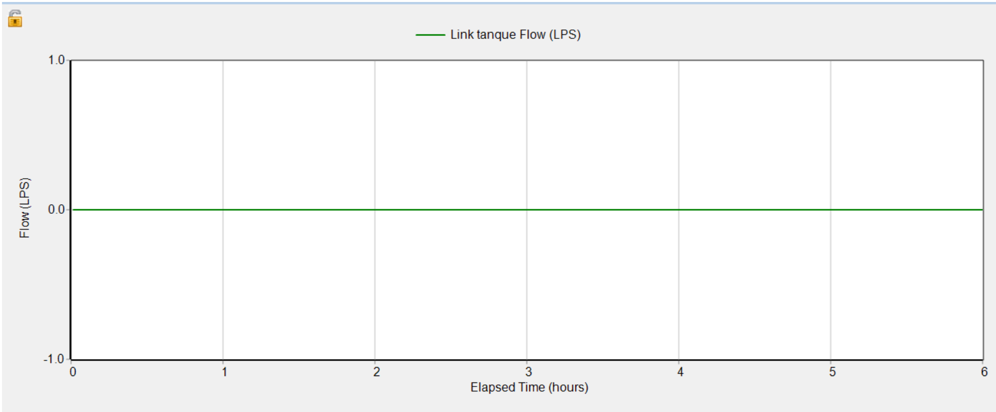


Figure 138. Discharged flow on short-term design storm. Alternative 4

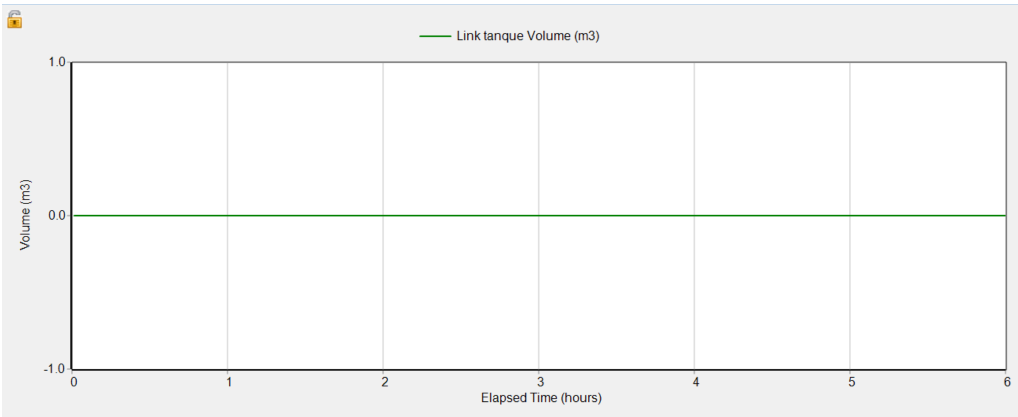


Figure 139. Discharged volume on short-term design storm. Alternative 4

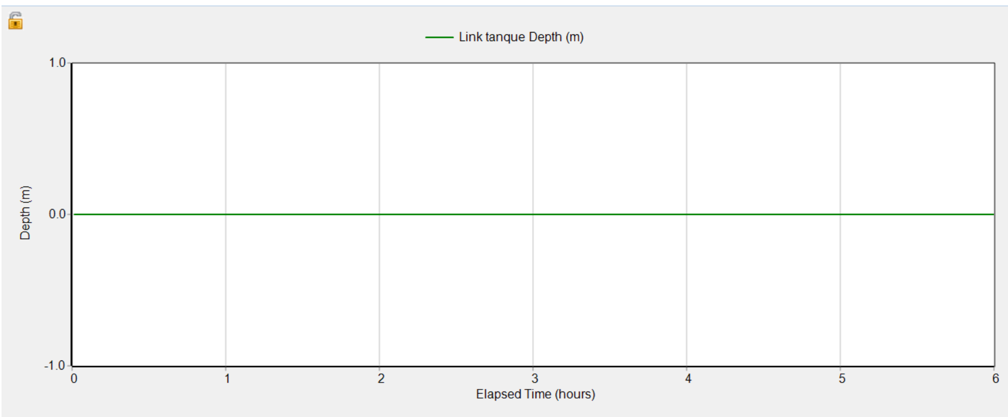


Figure 140. Depth on short-term design storm. Alternative 4

5.9.2.2 Medium-term design storm. Alternative 4

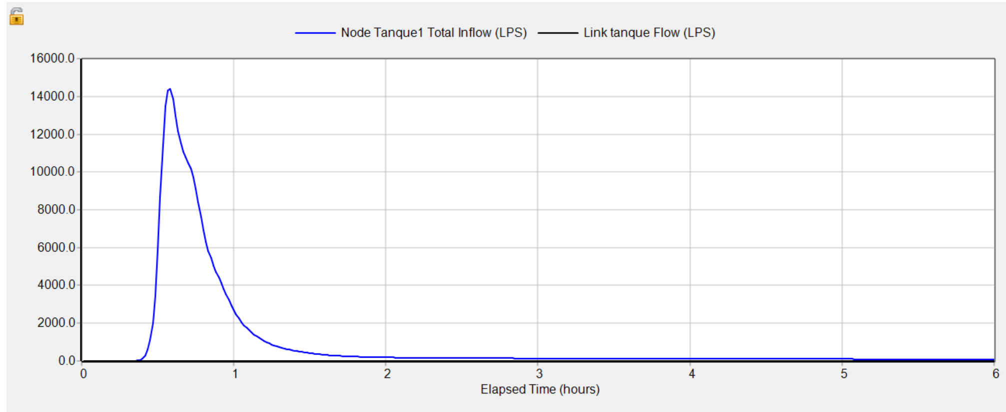


Figure 141. Comparison of inflow and outflow of the storm tank on medium-term design storm. Alternative 4

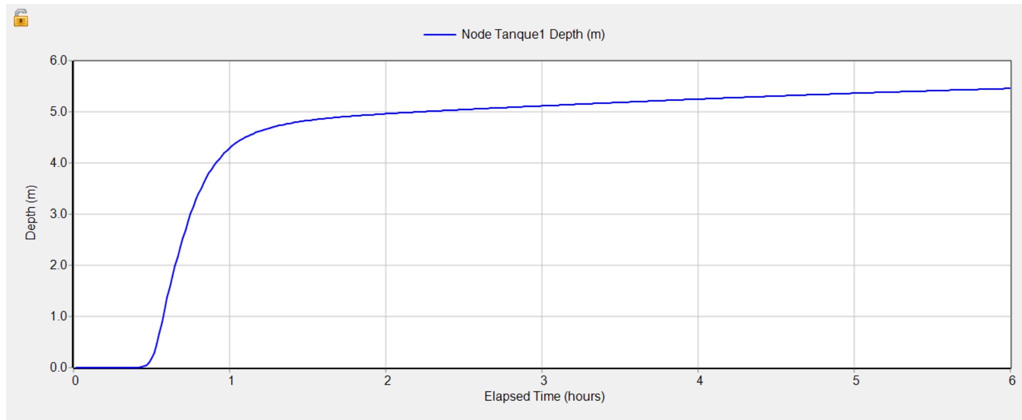


Figure 142. Inner depth of the storm tank medium-term design storm. Alternative 4

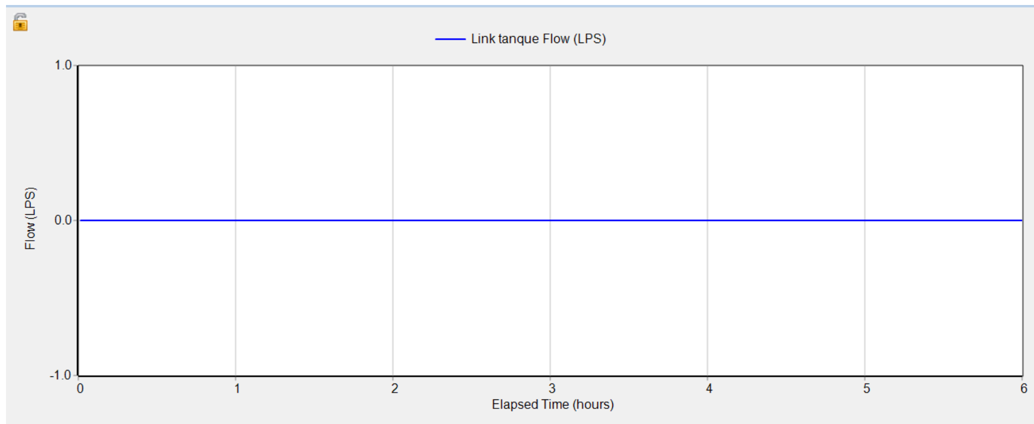


Figure 143. Discharged flow on medium-term design storm. Alternative 4

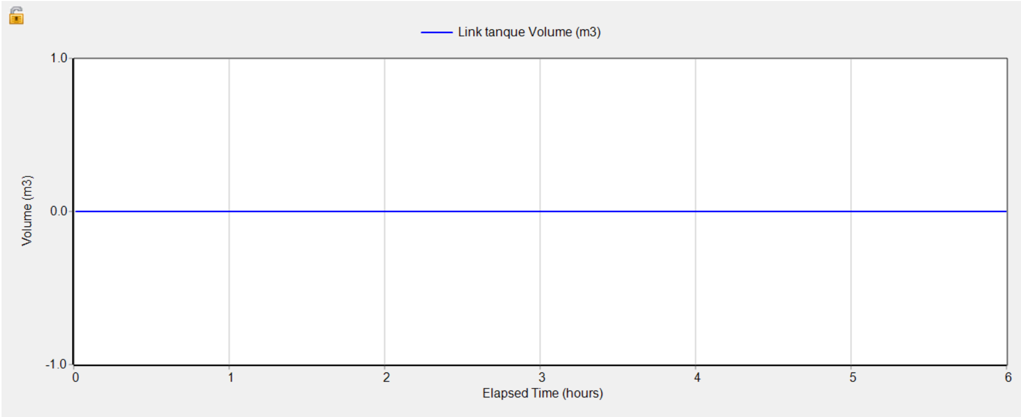


Figure 144. Discharged volume on medium-term design storm. Alternative 4

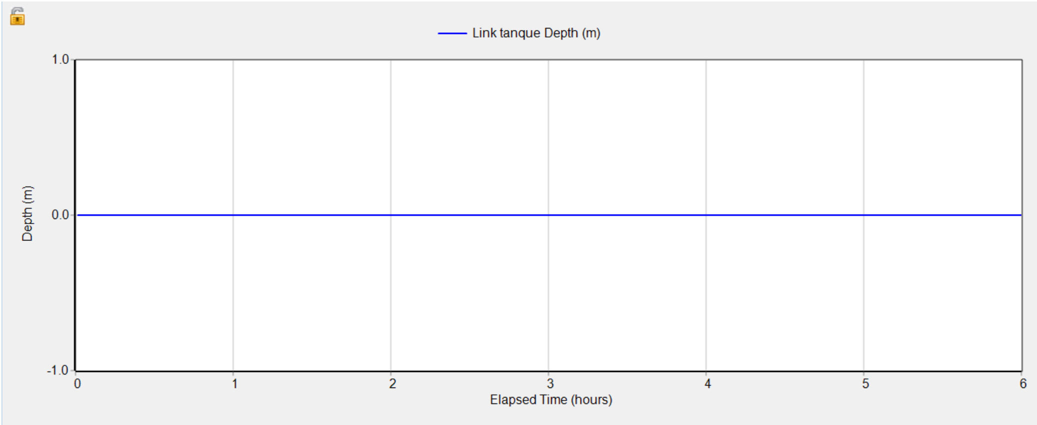


Figure 145. Depth on medium-term design storm. Alternative 4

5.9.2.3 Long-term design storm. Alternative 4

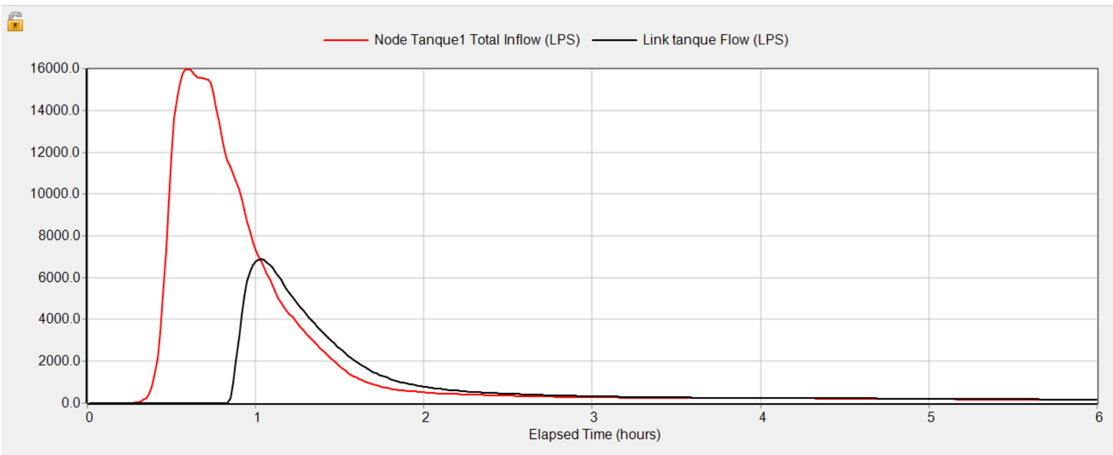


Figure 146. Comparison of inflow and outflow of the storm tank on long-term design storm. Alternative 4

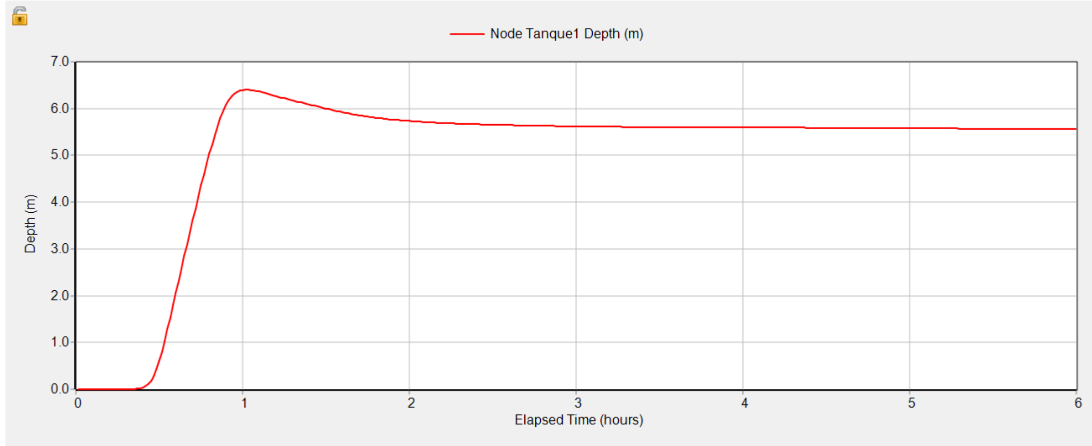


Figure 147. Inner depth of the storm tank long-term design storm. Alternative 4

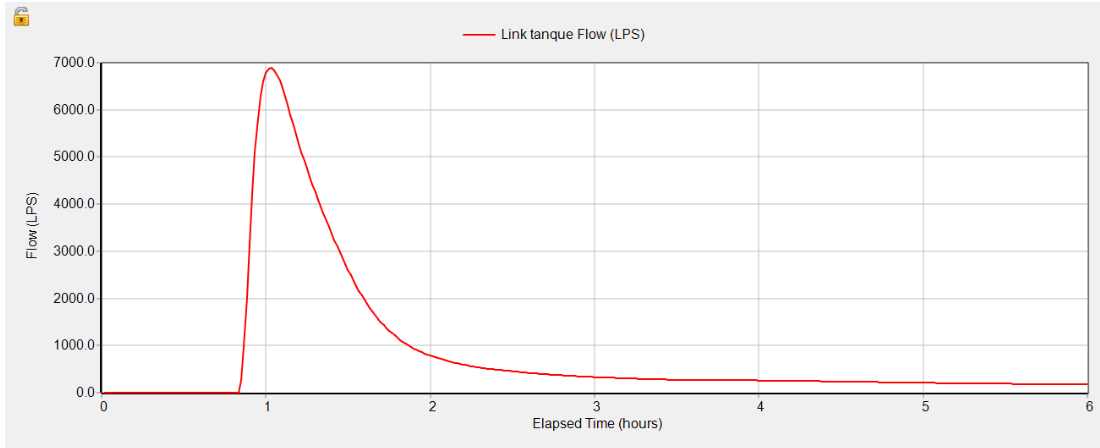


Figure 148. Discharged flow on long-term design storm. Alternative 4

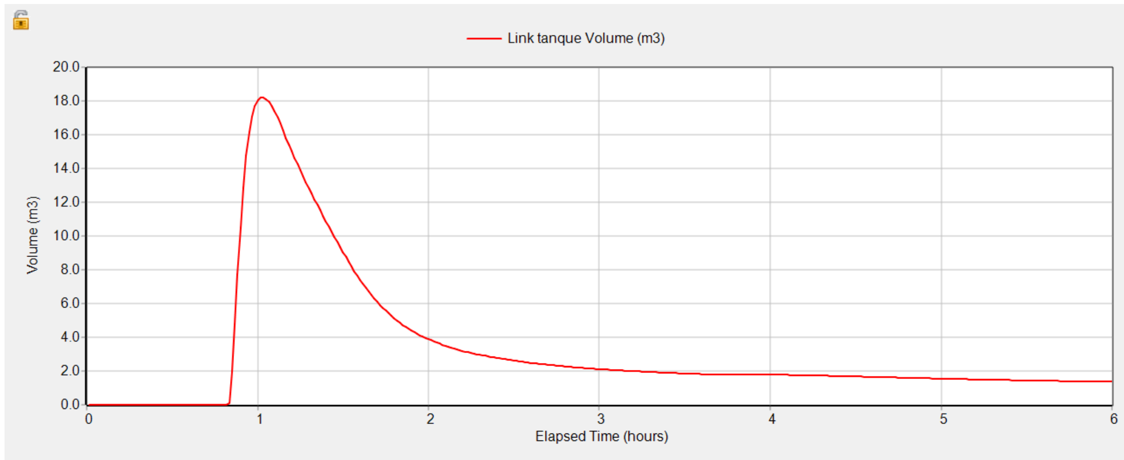


Figure 149. Discharged volume on long-term design storm. Alternative 4

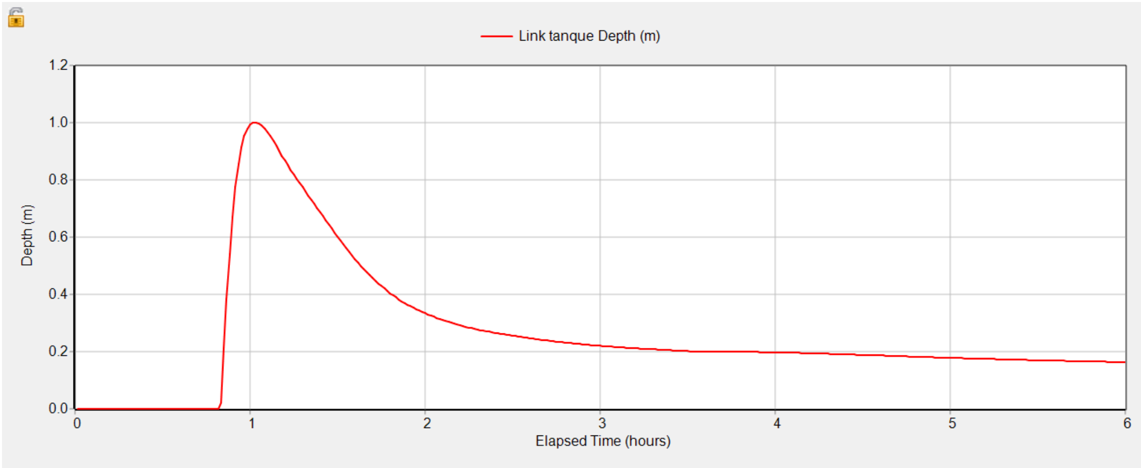


Figure 150. Depth on long-term design storm. Alternative 4

5.9.3 Stormwater tank emptying system. Alternative 4

As with the storm tank in alternative 2, it should be checked whether a pumping system is needed to empty the storm tank.

$$z, inf, tank = +50{,}24\ m$$

$$z, sewage = +60{,}50\ m$$

As foreseen, the pump to be used will not be the same as in scenario 2 because the characteristics of the emptying flow and the manometric head are different.

The flow rate chosen is the one that can empty the entire storm tank in half a day.

$$V_{tank} = \mathbf{23.396}\ \boldsymbol{m3}$$
$$t_{empty} = \mathbf{12}\ \boldsymbol{h}$$

$$Q_{empty} = \mathbf{542}\ \boldsymbol{l/s}$$

The manometric height between two points, A and B, is calculated using Bernoulli's equation:

$$H = (z_B - z_A) + \left(\frac{P_B}{\gamma} - \frac{P_A}{\gamma}\right) + \left(\frac{{V_B}^2}{2g} - \frac{{V_A}^2}{2g}\right) + \sum \frac{8 * f * L * Q^2}{g * \pi^2 * D^5} + \sum \frac{8 * k * Q^2}{g * \pi^2 * D^4}$$

In this rough calculation, friction losses and singular losses have been excluded.

$$H = \mathbf{10{,}26}\ \boldsymbol{m}$$

After defining the flow rate and manometric height to be supplied by the pump, each pump manufacturer has a quick selection table that allows the model that offers the best performance to be obtained from the whole range of pumps it presents.

In the following image, a sample selection table from the pump manufacturer's catalogue "Bombas ideal" is attached, showing the areas of best performance for each pump model:

Figure 151. Pump choice considering flow and manometric height. Source. Bombas ideal

Finally, a pump type **CPH 350-360**, of 50 Hz and 1450 RPM is chosen, whose characteristics can be obtained from the manufacturer's graph below:

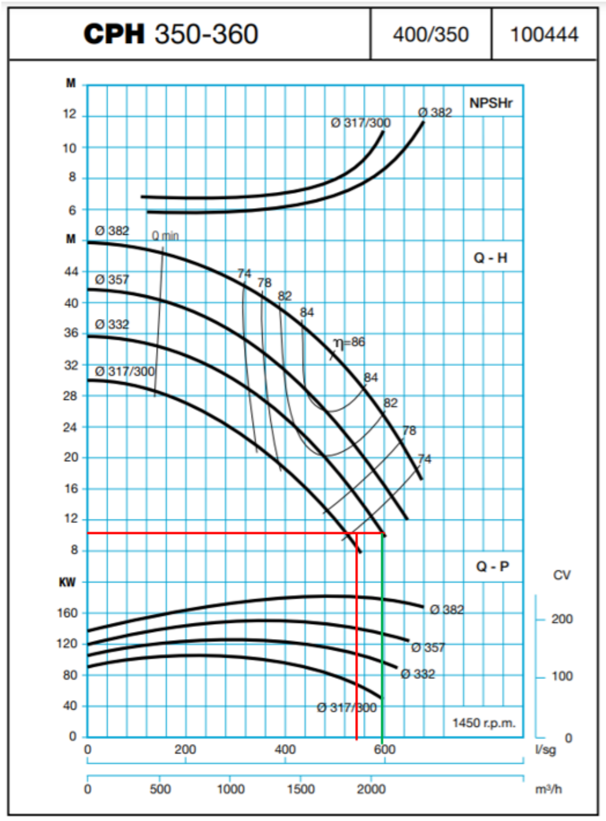


Figure 152. Specification pimp choices on CPH350-360 pump. Source: Bombas ideal

In red we identify the point at which our pumping needs are located and in green the exact point that the pump we have chosen offers us. Thus we obtain:

$$\boldsymbol{\eta = 72\%}$$
$$\boldsymbol{\emptyset = 332}$$

And a real emptying flow corresponding to:

$$\boldsymbol{Q, empty = 600\ l/s}$$

This will allow the tank to be emptied in **10,9 hours.**

5.10 Summary of modelling results

The most remarkable hydraulic results of all alternatives, i.e. peak flow and volume discharged, have been compiled in order to make a combined comparison.

Table 52. Comparison on alternatives' peak flow

Design storm	Peak flow (m3/s)			
	Alt 1	Alt 2	Alt 3	Alt 4
Short-term	24,49	**Short-term**	24,49	**Short-term**
Medium-term	29,71	**Medium-term**	29,71	**Medium-term**
Long-term	33,29	**Long-term**	33,29	**Long-term**

Starting from the base scenario, which is alternative 1, it has been possible to identify the flow reductions for each alternative:

Table 53. Comparison on alternatives' peak flow reduction

Design storm	Peak flow reduction			
	Alt 1	Alt 2	Alt 3	Alt 4
Short-term	-	**Short-term**	-	**Short-term**
Medium-term	-	**Medium-term**	-	**Medium-term**
Long-term	-	**Long-term**	-	**Long-term**

With regard to the volume discharged, the different solutions are compared in the table below:

Table 54. Comparison on alternatives' discharge volume

Design storm	Discharge volume (m3)			
	Alt 1	Alt 2	Alt 3	Alt 4
Short-term	25.945	0	13.101	0
Medium-term	40.208	0	20.667	0
Long-term	74.809	18.222	39.584	17.868

ANNEX Nº4: COMPARATIVE MULTI-CRITERIA ANALYSIS

SUSTAINABLE DRAINAGE SYSTEMS (SUDS) IN INDUSTRIAL AREAS:
APPLICATION TO THE INDUSTRIAL AREA IN QUART DE POBLET (VALENCIA)

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UNIVERSITAT POLITÈCNICA DE VALÈNCIA

Summary

TABLES INDEX

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1 OBJECT

The aim of this annex is to analyse and compare the proposals of the different scenarios presented and to find the optimum solution by carrying out a multi-criteria analysis. For this purpose, the "E2stormed" tool will be used, which is a Decision Support Tool that helps in the decision-making process by analysing the advantages and disadvantages of drainage systems for stormwater management, taking into account financial, hydraulic, energy, environmental and social criteria.

2 INTRODUCING E2STORMED

The E2Stormed software was developed as part of a European project of the same name within the MED Programme of the European Union, between 2013 and 2015, and coordinated by the IIAMA, Institute of Water and Environmental Engineering (Polytechnic University of Valencia), with the support of the University of Abertay Dundee.

Figure 1. E2Stormed logo. Source: Morales, A. et al., 2015

The programme is initially developed to achieve the objective of improvement of energy efficiency on the water cycle using innovative storm water management in Smart Mediterranean cities.

It is a software that functions as a Decision Support Tool to assist local authorities in decision making. Comparing different Decision Support Tools available, it was concluded that none of them included the criterion of energy efficiency. For this reason, E2stormed makes a comparison taking into account not only financial or hydraulic criteria, but also energy, environmental and social criteria.

The functions of the tool are:
- Define different scenarios of drainage systems
- Define the advantages and disadvantages of each scenario analysed.
- Compute and represent for each scenario the costs, energy consumed and CO2 emissions.
- Use the results to develop a basis of decision criteria and perform a multi-criteria analysis to choose the optimal option according to these criteria.

The general structure of the programme is clearly shown in the following diagram, which identifies the input data of the drainage systems to be analysed and the criteria used for the analysis.

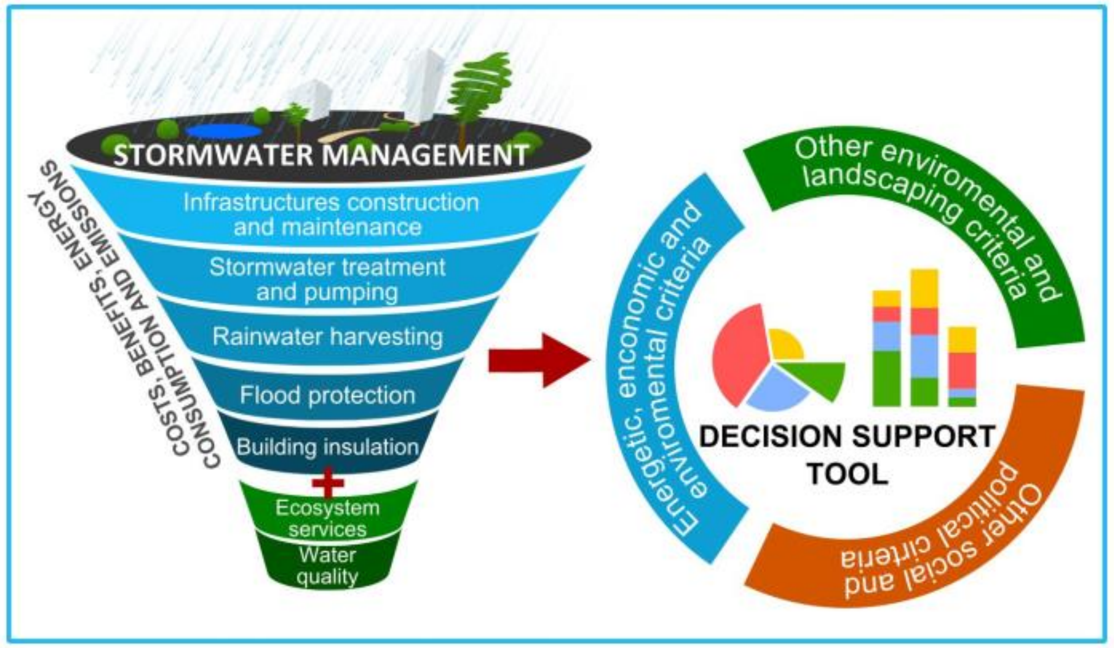

Figure 2. General structure of the E2stormed DST. Fuente: Morales et al. 2015

There are three headings to be completed for the comparison of solutions:
- General data
- New scenario
- New infrastructure

In the following, each of the sections of the programme will be explained and the values taken in each case will be indicated.

3 GENERAL DATA

The general data are the same for all scenarios. They include the context of the project in terms of both site and climatic characteristics. They are collected as follows:

- **Country:** Spain
- **Economic units' currency:** Euros (€)
- **Electricity price:** 0,1345 €/Kwh
  Corresponds to Iberdrola's price in Spain as of February 2021
- **Electricity emissions:** 0,238 $kgCO_2e$/Kwh
  Software default value
- **Analysis period:** 50 years
  Corresponds to lifespan of the project
- **Economic discount rate:** 3%
- **Return period:** 15 years
- **Rainfall distribution**
  Obtained from the pluviometric analysis carried out in the appendix of the initial data

Table 1. Average monthly rainfall in the study area

Month	Rainfall (mm)
1	33,15
2	29,76
3	35,29
4	27,23
5	34,35
6	20,99
7	10,33
8	14,91
9	55,21
10	70,09
11	50,20
12	44,13

- **Temperature distribution**
  Obtained from the State Meteorological Agency (AEMET)

Table 2.Temperature distribution by time frames

Time frame	Average winter temperature	Average summer temperature
	ºC	ºC
0:00 – 2:00	8	20
2:00 – 4:00	6	20
4:00 – 6:00	5	18
6:00 – 8:00	5	20
8:00 – 10:00	8	25
10:00 – 12:00	10	30
12:00 – 14:00	12	34
14:00 – 16:00	12	33
16:00 – 18:00	11	31
18:00 – 20:00	10	28
20:00 – 22:00	10	25
22:00 – 24:00	9	22

4 NEW SCENARIO

In the new scenario option, all aspects representing each scenario or proposal are added. The four proposals are presented and the discharge point, which is common to all of them, is indicated.

- **Discharge point:** directly to the natural environment (option to choose between unitary network, directly to the natural environment or separate network)

Subsequently, the opportunity is given to add, for each proposal, data related to the following aspects:

- Water supply
- Stormwater runoff
- Conveyance and treatment
- Water quality
- Flood protection
- Building insulation
- Ecosystem services

4.1 Water supply

In this section, data on the water supply system is introduced in order to qualify the costs and benefits obtained by the water consumed and saved due to rainwater management.

On the one hand, the reuse of rainwater due to the rainwater harvesting system makes it possible to reduce the consumption of water used, which means an economic saving and a reduction in the energy consumed for the purchase and treatment of drinking water. On the other hand, some drainage infrastructures may require drinking water for irrigation or cleaning. This increase in the volume of water consumed increases costs and electricity consumption.

4.1.1 Water supply cost

- Water cost: **0,525 €/m3**
  Rate for water consumption over 20 m3/month for the municipality of Quart de Poblet obtained from the 2018 rate report (Ayuntamiento Quart de Poblet, 2018)

4.1.2 Energy consumed in water acquisition

The supply of the municipality of Quart de Poblet is carried out by extracting groundwater from a well. This well is located in Avenida San Onofre and has a depth of 38 metres. The water undergoes chlorination treatment to meet the quality requirements set by the regulations. Therefore, for all scenarios these values are the same.

Putting this data together with the default values of the system in terms of mechanical efficiency and efficiency of the electrical system, we finally obtain:

- Energy consumed in obtaining water: **0,188 Kwh/m3**
- Emissions in obtaining water**: 0,0447 kgCO2e/m3**

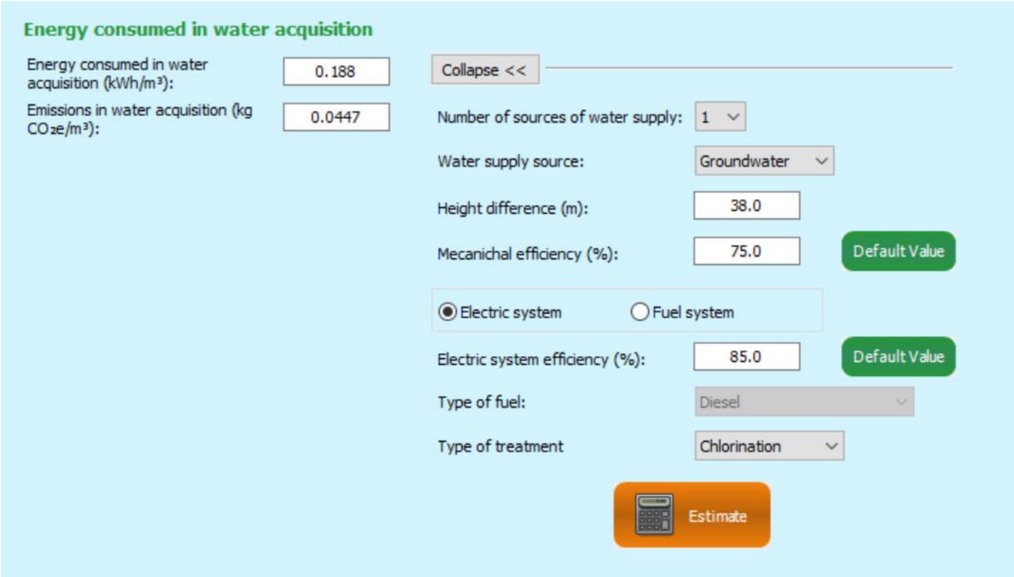

Figure 3. Estimation on energy consumed in water acquisition. Source: E2Stormed

4.1.3 Energy consumed in water conveyance

The water extracted from the well is taken to an elevated reinforced concrete tank with a maximum height of 27 metres and is located 100 metres from where the water is collected. The pipe of this tank is 400 mm of polyethylene, with a Manning's roughness of 0.002.

These data give us the following values for water conduction:
- Energy consumed in wáter conveyance: **0,177 Kwh/m3**
- Emissions in water conveyance: **0,0279 kgCO2e/m3**

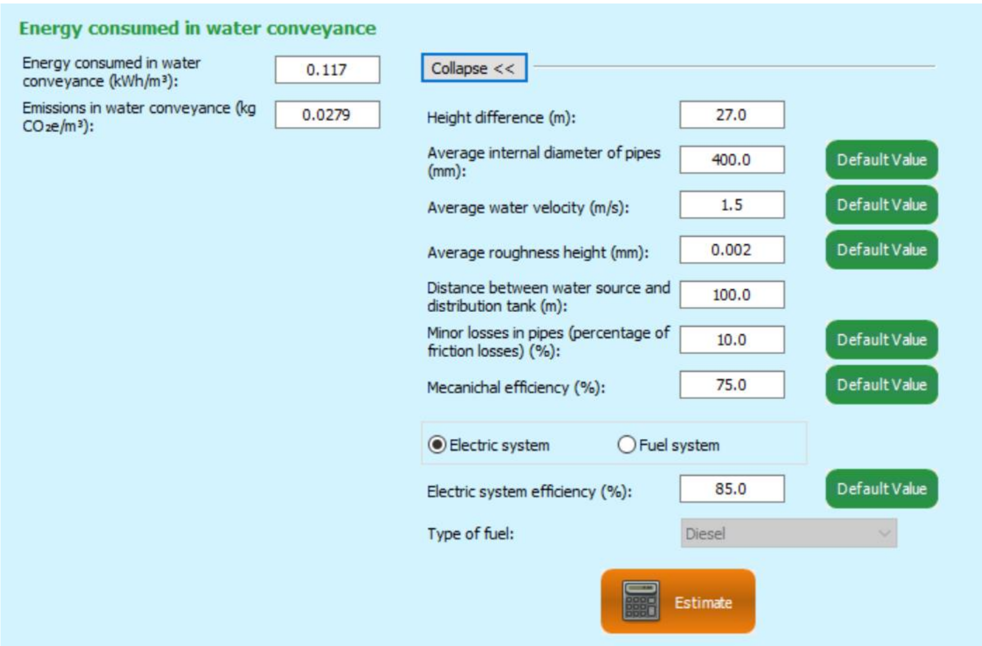

Figure 4. Estimation on energy consumed in water conveyance. Source: E2Stormed

4.1.4 Energy consumed in water distribution

The water is distributed by gravity, as the tank is elevated. Therefore, energy and emissions are zero.
- Energy consumed in water distribution: **0 Kwh/m3**
- Emissions in water distribution: **0 kgCO2e/m3**

4.1.5 Water supply network

The Town Hall of Quart de Poblet wants to launch the pilot of the Fuga-0 project. An advanced procedure that allows the early detection and localisation of water leaks in water supply networks. By using a series of virtual micro-sectors of the network and monitoring the hydraulic values through flow meters, the system can detect any anomaly in the water flow and locate it geographically. As a result, network losses are estimated to be very low, at around 10%.
- Losses in the network: **10%**.

4.1.6 Irrigation or cleaning of drainage structures

The need for irrigation or cleaning varies throughout the year. Estimating a ratio of 6l/day/m2 for the green drainage systems, the following values are obtained for each solution. As the rest of the drainage systems are common to all the alternatives, they are not taken into account in this section.

Table 3. Parameters for irrigation or cleaning drainage structures

Parameters	Alt 1	Alt 2	Alt 3	Alt 4
Green drainage area (m2)	0	0	62.037	62.037
Volume of consumed water (m3/year)	0	0	135.861	135.861

4.1.7 Rainwater reuse by harvesting systems

It is not foreseen to reuse rainwater for irrigation or any other purposes.
- Volume of reused water: **0 m3/year**

4.2 Stormwater runoff

This section introduces all the data related to qualitative hydraulic aspects of the drainage systems.

Some drainage infrastructures produce a significant reduction in runoff volume and flow. The reduction in volume can lead to a reduction in the frequency of discharges. One of the main objectives of the work is to reach a threshold flow corresponding to the flow in a natural state, so this section is of particular importance for decision-making.

The following parameters have been presented for each scenario:

- **Runoff volume generated per year (m3/year):** depends on rainfall, drained area and land use. It corresponds to the total volume at the discharge point if there would be no influence of aquifer recharge or evapotranspiration. For the comparison, the data of the long-term design storm of each alternative has been obtained, as it is the most disadvantageous alternative and the one that will give the highest volumes.
- **Aquifer recharge and evapotranspiration ratio (m3/year):** this is the difference between the volume of runoff generated and the volume that reaches the previous point. An equal value for all systems of 141 m3/year is considered, based on the modelling results in SWMM.
- **Design peak storm flow (m3/s):** being one of the most important criteria for decision making, this section is only completed if it is a decision criterion.
- **Unitary System Discharge:** In case there is a unitary network where wastewater and stormwater are collected in the same collector. In the case of the study, a separate network has been proposed since, in unitary networks, when there are large episodes of rainfall where the wastewater treatment plant cannot treat the entire volume it receives, the combined water is discharged into the receiving environment without being treated and this leads to greater point source pollution of the water masses. Being a separative network, the discharge is null, 0 m3/year.

Table 4. Stormwater runoff parameters

Parameters	Alt 1	Alt 2	Alt 3	Alt 4
Runoff volume generated (m3/year)	74.950	18.222	39.584	17.868
Peak design storm flow (m3/s)	33,29	6,88	15,91	6,88

4.3 Conveyance and treatment

4.3.1 Stormwater pumping

The addition of a pumping system in sustainable drainage systems involves high cost and energy consumption that must be considered for the analysis. The proposed drainage systems are designed to operate by gravity to avoid pumping systems. However, solutions with a buried storm tank, upstream of the discharge point, will have to pump the water from the tank up to the height where the separate sewage network is located.

The proposals in which stormwater will be pumped are limited to alternatives 2 and 4.
Knowing the lower elevation of the storm tank and the elevation of the separative network into which this water will be discharged, the programme gives us an estimate of the cost, consumption and emissions.

Table 5. Differences on pumping system heights

Parameters	Alt 1	Alt 2	Alt 3	Alt 4
Lower height of storm tank (m)	-	46,64	-	50,24
Separate network height (m)	-	60,50	-	60,50
Height difference (m)	-	13,96	-	10,26

Table 6. Results for treatment and pumping stormwater

Parameters	Alt 1	Alt 2	Alt 3	Alt 4
Total cost (€/year)	0	146,32	0	105,42
Total energy consumed (kWh/year)	0	1.087,9	0	784,41
Total emissions (Kg CO2e/year)	0	258,75	0	185,83

4.3.2 Stormwater treatment

It is not foreseen to treat the water before discharge into the environment, therefore this section will not be filled.

4.4 Water quality

The water quality section provides information on each component of the drainage infrastructure that can be used as a guide to estimate a quantitative value for runoff quality.

First, the land use of the study area and the degree of vulnerability of the receiving waters should be indicated.

- Drainage catchment characteristics: **industrial areas**
- Sensitivity of receiving water: **medium**

In order to determine the water quality, all elements of each drainage system must be analysed. E2Stormed provides guidance parameters for the qualitative evaluation of the efficiency of pollutant removal according to the reduction of total suspended solids, nutrients and heavy metals.

Type of drainage infrastructure	Total suspended solids	Nutrients	Heavy metals
Conventional drainage networks	Low	None	Low
Convertional roofs	None	None	None
Standard pavement	None	None	None
Structural detention facilities	Medium	None	Low
Rain harvesting systems	High	Low	Medium
Water butts	Low	Low	Low
Green roofs	High	Low	Medium
Permeable pavements	High	High	High
Soakaways	Medium	Low	Medium
Infiltration trenches	High	Low	High
Geocellular systems	Low	None	Low
Bioretention areas	High	Low	High
Rain gardens	High	Low	High
Filter strips	Medium	Low	Medium
Filter drains	High	Low	High
Vegetated swales	High	Low	Medium
Infiltration basins	High	Medium	High
Detention basins	Medium	Low	Medium
Retention ponds	High	Medium	High
Constructed wetlands	High	Medium	High

Figure 5. Qualitative evaluation of pollutants removal efficiency of each drainage infrastructure. Source: E2Stormed

Knowing the elements of each alternative, the elements of the drainage network and their mitigation of pollutants have been identified. The following table gives a general summary of the parameters chosen for each alternative, followed by a detailed description of the elements of the drainage system for each alternative.

Table 7. Quality parameters for each alternative

Parameters	Alt 1	Alt 2	Alt 3	Alt 4
Total suspended solids removal efficiency	Low	Medium	High	High
Nutrients removal efficiency	None	None	Medium	Medium
Heavy metals removal efficiency	Low	Low	High	High
Average water quality	**Low**	**Low**	**High**	**High**

4.4.1 Water quality. Alternative 1

Scenario 1 is a network of drainage systems consisting of pipe network, standard pavement and standard garden.

Table 8. General water quality in alternative 1

Infrastructure	Total suspended solids removal efficiency	Nutrients removal efficiency	Heavy metals removal efficiency
Pipe network	Low	None	Low
Standard pavement	None	None	None
Standard garden	High	Low	High
GENERAL WATER QUALITY	**LOW**	**NONE**	**LOW**

The average water quality determined for alternative 1 is **LOW**.

As expected, this alternative does not allow for good water quality as it does not have elements that act directly on the reduction of pollutants.

4.4.2 Water quality. Alternative 2

Scenario 2 has the same elements of the drainage system as alternative 1, but with the addition of a structural detention facility.

Table 9. General water quality in alternative 2

Infrastructure	Total suspended solids removal efficiency	Nutrients removal efficiency	Heavy metals removal efficiency
Pipe network	Low	None	Low
Standard pavement	None	None	None
Standard garden	High	Low	High
Structural detention facility	Medium	None	Low
GENERAL WATER QUALITY	**MEDIUM**	**NONE**	**LOW**

The installation of the storm tank increases the water quality in terms of total suspended solids from low to medium. Even so, the average water quality determined for scenario 2 is still in the **LOW** category.

4.4.3 Water quality. Alternative 3

Scenario 3, which introduces sustainable drainage systems is based on scenario 1 by adding elements such as permeable pavement, bioretention area and infiltration basin.

Table 10. General water quality in alternative 3

Infrastructure	Total suspended solids removal efficiency	Nutrients removal efficiency	Heavy metals removal efficiency
Pipe network	Low	None	Low
Standard pavement	None	None	None
Standard garden	High	Low	High
Permeable pavement	High	High	High
Bioretention area	High	Low	High
Infiltration basin	High	Medium	High
GENERAL WATER QUALITY	**HIGH**	**MEDIUM**	**HIGH**

Having already commented on the benefits of sustainable drainage systems in the reduction of pollutants, the overall water quality has increased considerably.

In this scenario the average water quality determined is **HIGH.**

4.4.4 Water quality. Alternative 4

The last scenario adds to alternative 3 the characteristics of a structural detention facility.

Table 11. General water quality in alternative 4

Infrastructure	Total suspended solids removal efficiency	Nutrients removal efficiency	Heavy metals removal efficiency
Pipe network	Low	None	Low
Standard pavement	None	None	None
Standard garden	High	Low	High
Permeable pavement	High	High	High
Bioretention area	High	Low	High
Infiltration basin	High	Medium	High
Structural detention facility	Medium	None	Low
GENERAL WATER QUALITY	**HIGH**	**MEDIUM**	**HIGH**

The average toilet quality for scenario 4 remains in the **HIGH** category, as the structural detention facility increases the total suspended solids removal efficiency and in the previous case it was already at its highest value, which corresponds to high.

4.5 Flood protection

The flood risks of the study area were presented in the initial data annex based on the maps provided by PATRICOVA. All the proposals in the study are designed for the same flood protection, the design return period being 15 years, but because each has a different discharge flow they will have different consequences.

Analysing the order of magnitude of the results, it has been considered that the difference in flood damage that may be caused to a greater extent by one solution than the other will not be taken into account in the multi-criteria analysis.

The first reason is that such an analysis requires an in-depth analysis of the flood risk and its economic consequences. There are different softwares that allow this analysis, among them we can highlight the software "ipresas", a spinoff developed by the Polytechnic University of Valencia that works as a Decision Support Tool to analyse the risk of different scenarios. However, this would require an exhaustive analysis of the data, the context and damage estimates that are beyond the scope of this project.

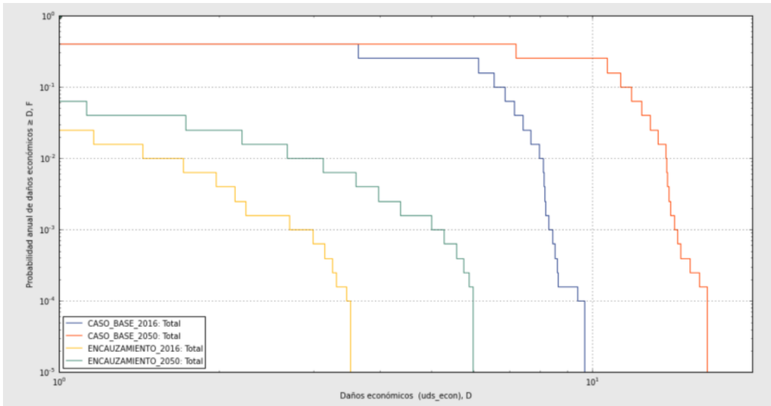

Figure 6. Example on flood risk analysis by ipresas software. Source:ipresas

Secondly, the design return period is of considerably high frequency. Even in the PATRICOVA flood hazard maps, the highest frequency that gives higher hazard values is 25 years, higher than the design return period.

For these two reasons, an analysis of comparative flood protection will not be carried out.

4.6 Building insulation

This section considers the advantages of green roofs and how they can improve the thermal insulation of a building, which reduces the energy consumption for heating and air conditioning. It allows estimating the economic and energy advantages of insulating the building compared to a green roof or a conventional roof.

As the proposed alternatives do not make use of green roofs, the benefits, consumption and emissions avoided are zero.

- Building insulation benefits: **0 €/year**
- Avoided energy consumption: **0 Kwh/year**
- Avoided emissions: **0 kg CO^2/year**

4.7 Ecosystem services

Ecosystem services are those aspects of drainage infrastructure from which benefits such as carbon dioxide capture by vegetation are obtained from drainage infrastructure, in addition to making an evaluation of the overall ecosystem services.

The list of ecosystem services provided by the selected infrastructure and indicated by the programme are:
- Aesthetics
- Air quality improvement
- Amenity Community education and engagement
- Community space improvement
- Enhancement of quality of life
- Firm dry surfaces to park and walk on after heavy rain
- Food growing
- Habitat provision and enrich biodiversity
- Improved community cohesion
- Improvements to public health
- Noise attenuation
- Provision of educational opportunities
- Recreational use
- Reduction of greenhouse gas emissions
- Regulation of urban microclimates
- Visual and landscape benefits

The programme also offers for each element a recommended value to be taken from the ecosystem service, classifying it as none, low, medium and high. For the elements used in the drainage system proposals, we have:

Table 12. Ecosystem services for each infrastructure

Infrastructure	Ecosystem services
Pipe network	None
Standard pavement	None
Standard garden	High
Permeable pavement	Medium
Bioretention area	High
Infiltration basin	Low
Structural detention facility	None

Considering the elements that make up each alternative, the following evaluation of global ecosystem services has been proposed based on the options provided of very low, low, medium, high and very high:

Table 13. Evaluation of global ecosystem services

Parameters	Alt 1	Alt 2	Alt 3	Alt 4
Global ecosystem services	Very low	Very low	Medium	Medium

For the unit reduction of carbon dioxide in landscaped area, the default value provided by the programme of **0.068** is taken as the default value.

In this case, the additional vegetated area added in the solutions with sustainable drainage systems, i.e., rain gardens installed on part of the conventional paving in alternatives 1 and 2, will be considered.

Table 14. Carbon reduction calculation with vegetated drainage

Parameters	Alt 1	Alt 2	Alt 3	Alt 4
Total vegetated drainage system area (m2)	0	0	26.631	26.631
Carbon dioxide reduced by vegetation (Kg/CO2e/year)	0	0	1.810,9	1.810,9

5 NEW INFRASTRUCTURE

This section includes all the infrastructures of each alternative, including the cost, the energy consumed and the emission of pollutants for the construction and maintenance process.

All the values that could be determined from the proposal of alternatives will be taken and, in the event that some data is unknown, it will be estimated using the default values offered by the software. These values will be used mainly for the unit prices.

The following table shows the maintenance and construction costs for each alternative and then details the costs for each scenario, divided by elements within the drainage system.

Table 15. Construction and maintenance costs for each alternative

Infrastructure	CONSTRUCTION			MAINTENANCE		
	Cost (€)	Energy consumed (kWh)	Emissions (kgCO2e)	Cost (€/year)	Energy consumed (kWh/year)	Emissions (kgCO2e/year)
Alternative 1	20.203.700	57.042.850	17.962.588	295.348	6.055	1.594
Alternative 2	51.334.200	109.921.495	34.712.042	388.740	6.063	1.596
Alternative 3	22.252.072	53.657.674	16.895.614	550.329	5.380	1.417
Alternative 4	33.950.072	73.527.662	23.189.606	585.423	5.388	1.419

5.1 Alternative 1

Table 16. Infrastructure properties of alternative 1

Infrastructure	Dimension	Lifespan (years)
Pipe network	10.725,2 m	35
Standard pavement	301.614,6 m2	35
Standard garden	59.558,6 m2	30

Table 17. Construction and maintenance costs of alternative 1

Infrastructure	CONSTRUCTION			MAINTENANCE		
	Cost (€)	Energy consumed (kWh)	Emissions (kgCO2e)	Cost (€/year)	Energy consumed (kWh/year)	Emissions (kgCO2e/year)
Pipe network	2.145.040,0	346..638,46	102.532,91	10.725,2	4,012	1,072
Standard pavement	15.080.730,0	49.669.892,32	15.717.136,8	135.726,57	124,657	31,233
Standard garden	2.977.930,0	7.027.319,2	2.142.918,4	148.896,5	5.926,578	1.561,39

5.2 Alternative 2

Table 18. Infrastructure properties of alternative 2

Infrastructure	Dimension	Lifespan (years)
Pipe network	10.725,2 m	35
Standard pavement	301.614,6 m2	35
Standard garden	59.558,6 m2	30
Structural detention facility	62.261,0 m3	50

Table 19. Construction and maintenance costs of alternative 2

Infrastructure	CONSTRUCTION			MAINTENANCE		
	Cost (€)	Energy consumed (kWh)	Emissions (kgCO2e)	Cost (€/year)	Energy consumed (kWh/year)	Emissions (kgCO2e/year)
Pipe network	2.145.040,0	346.638,5	102.532,9	10.725,2	4,012	1,072
Standard pavement	15.080.730,0	49.669.892,3	15.717.136,8	135.726,57	124,657	31,233
Standard garden	2.977.930,0	7.027.319,2	2.142.918,4	148.896,5	5.926,578	1.561,39
Structural detention facility	31.130.500,0	52.877.644,7	16.749.454,2	93.391,5	8,024	2,144

5.3 Alternative 3

Table 20. Infrastructure properties of alternative 3

Infrastructure	Dimension	Lifespan (years)
Pipe network	10.725,2 m	35
Standard pavement	257.568,34 m2	35
Standard garden	24.153,21 m2	30
Permeable pavement	44.046,25	30
Bioretention area	26.631,6 m2	30
Infiltration basin	21.243,23 m3	50

Table 21. Construction and maintenance costs of alternative 3

Infrastructure	CONSTRUCTION			MAINTENANCE		
	Cost (€)	Energy consumed (kWh)	Emissions (kgCO2e)	Cost (€/year)	Energy consumed (kWh/year)	Emissions (kgCO2e/year)
Pipe network	2.145.040,0	346.638,5	102.532,9	10.725,2	4,012	1,072
Standard pavement	12.878.417,0	42.416.354,2	13.421.886,2	115.905,8	107,039	26,829
Standard garden	1.207.660,5	2.849.837,2	869.032,5	60.383,0	2.432,066	640,847
Permeable pavement	2.642.775,0	4.060.183,3	1.284.829,1	44.046,3	69,689	19,763
Bioretention area	1.997.370,0	3.651.991,4	1.127.049,3	213.052,8	2.676,682	705,286
Infiltration basin	1.380.809,9	332.668,9	90.283,7	106.216,2	90,872	23,387

5.4 Alternative 4

Table 22. Infrastructure properties of alternative 4

Infrastructure	Dimension	Lifespan (years)
Pipe network	10.725,2 m	35
Standard pavement	257.568,34 m2	35
Standard garden	24.153,21 m2	30
Permeable pavement	44.046,25	30
Bioretention area	26.631,6 m2	30
Infiltration basin	21.243,23 m3	50
Structural detention facility	23.396,0 m3	50

Table 23. Construction and maintenance costs of alternative 4

Infrastructure	CONSTRUCTION			MAINTENANCE		
	Cost (€)	Energy consumed (kWh)	Emissions (kgCO2e)	Cost (€/year)	Energy consumed (kWh/year)	Emissions (kgCO2e/year)
Pipe network	2.145.040,0	346.638,5	102.532,9	10.725,2	4,012	1,072
Standard pavement	12.878.417,0	42.416.354,2	13.421.886,2	115.905,8	107,039	26,829
Standard garden	1.207.660,5	2.849.837,2	869.032,5	60.383,0	2.432,066	640,847
Permeable pavement	2.642.775,0	4.060.183,3	1.284.829,1	44.046,3	69,689	19,763
Bioretention area	1.997.370,0	3.651.991,4	1.127.049,3	213.052,8	2.676,682	705,286
Infiltration basin	1.380.809,9	332.668,9	90.283,7	106.216,2	90,872	23,387
Structural detention facility	11.698.000,0	19.869.988,8	6.293.991,9	35.094,0	8,024	2,144

6 ANALYSIS RESULTS

The results provided by the programme can be categorised into two different types of results. The first type provides results that evolve over the lifetime of the project in terms of cost, energy consumption and emissions and the second type provide the result of the established multi-criteria analysis.

6.1 Temporal variation graphs

The results of the comparison are presented below in terms of temporal variation graphs throughout its lifespan.

It should be noted that for the estimation of financial costs and benefits, use is made of the application of the interest rate for the capitalization period. This means that the financial evaluation of the costs and benefits of each action is made with its present value by applying the following equation:

$$PV = \frac{FV}{(1+i)^n}$$

where:
- PV (€) is the present value of the cost or benefit
- FV (€) is the future quantity of money to be discounted
- n (years is the number of periods compounded between the current date and the future date.
- i (adimensional) is the interest rate for a compounding period, estimated at 3% in the general data.

6.1.1 Cumulative costs on present value

The cumulative costs present value of the four alternatives are compared and their order of preference in terms of their economics is identified.

The costs that are included in these results are:
- Construction of infrastructures
- Maintenance of infrastructures
- Water reuse
- Conveyance and treatment
- Flood protection
- Building insulation
- Co2e reduction
- Other costs and benefits

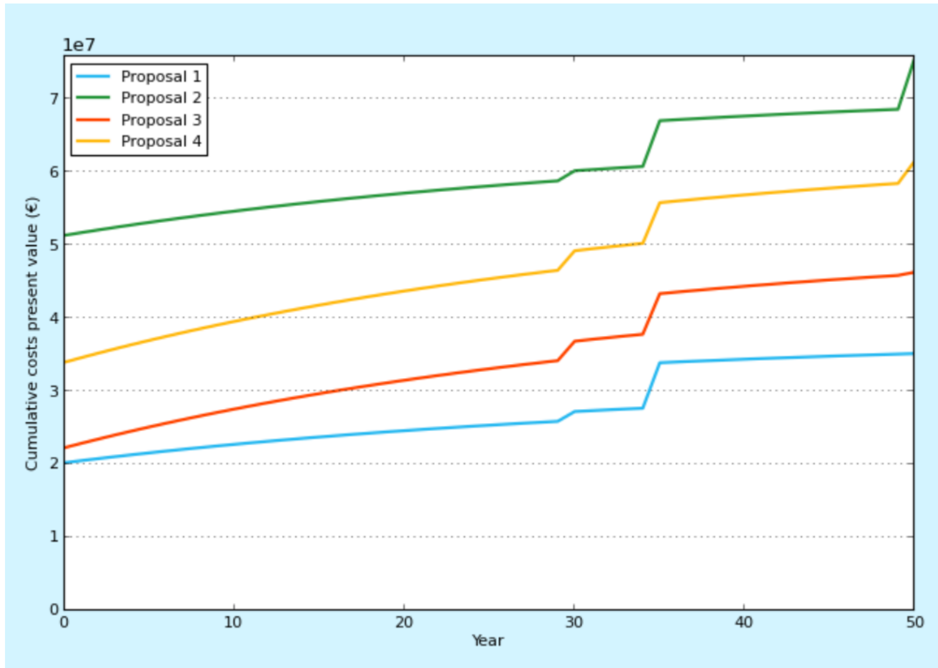

Figure 7. Cumulative total costs on present value during the lifespan. Source: e2stormed

The most economical solution is proposal 1, as expected since the other proposals are a modification of proposal 1 and therefore their construction and maintenance cost would be higher. Given that, as will be shown below, the construction and maintenance costs are of a much higher magnitude than the other costs, the difference in construction and maintenance cost plays a crucial role in the economics of the solutions.

Also, the most expensive solution is proposal 2 with the larger storm tank due to the construction cost of the tank and the maintenance cost of the water extraction pumping.

Between the two solutions that propose sustainable drainage systems, the solution that incorporates a storm tank is located at the top of the cost graph, due to the storm tank construction and maintenance costs.

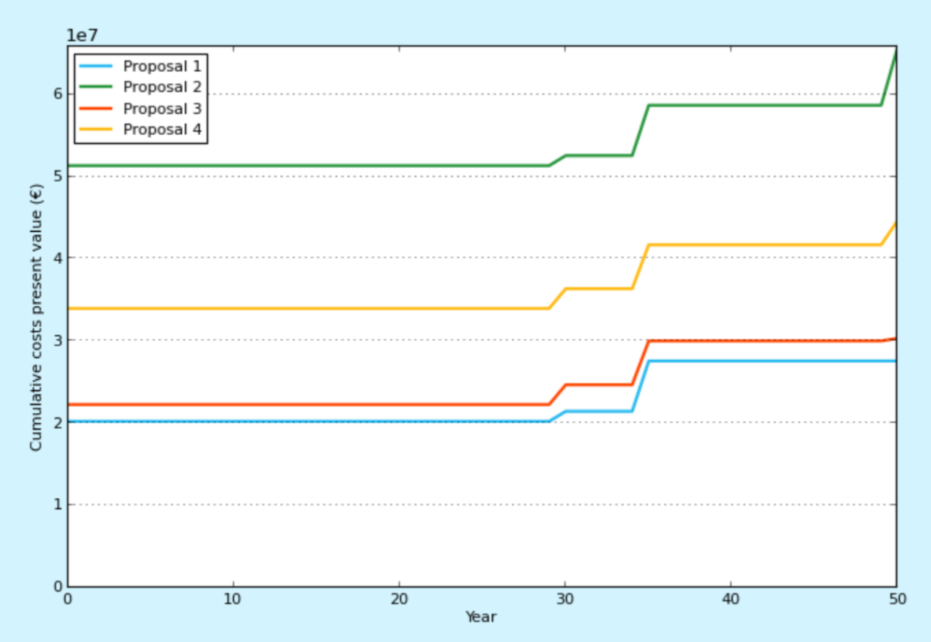

Figure 8. Cumulative construction costs on present value during the lifespan. Source: e2stormed

Construction costs follow the order of economy of total costs. The reason for this is their enormous impact on costs. It is interesting to note that the graph shows the steps that correspond to the lifespan of the elements in 30, 35 and 50 years. Not only for sustainable drainage systems, but also for standard systems such as standard pavements, standard garden and the general pipe network.

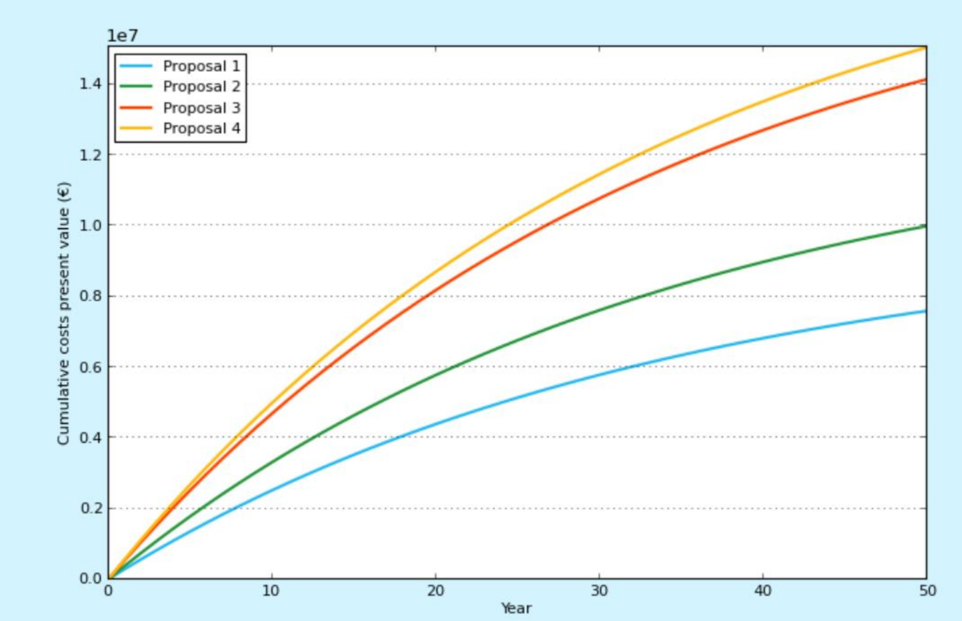

Figure 9. Cumulative maintenance costs on present value during the lifespan. Source: e2stormed

From the results of the cumulative maintenance costs, we can highlight two things, firstly the positive sloping trend that reduces with time and secondly the fact that the order of economic preference of the proposals is modified.

In terms of maintenance costs, the most economical alternatives are those with a conventional drainage system, with the proposals for sustainable drainage systems being the most expensive. This is mainly justified by the importance of the maintenance of the vegetation and cleaning of the SuDS, which require greater attention than in the case of a conventional system which will ne justified on monitoring and maintenance annex.

6.1.2 Cumulative energy consumption

E2stormed also provides the comparative results of energy consumption throughout the project. The results identify proposals 2 and 4 as those with the highest energy consumption. This is mainly due to the implementation of a pumping system to lift the water from the storm tank to the level where the wastewater network is located. Proposal 2 consumes more energy because the tank is larger and must draw more water from a greater depth. The comparative cumulative emissions of each proposal are presented in the graph below.

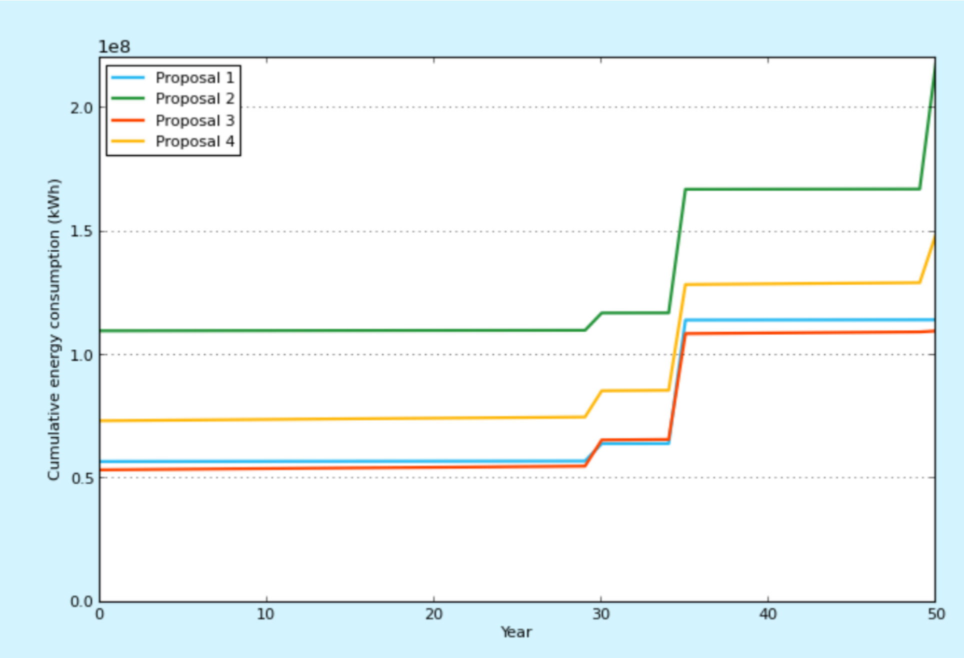

Figure 10. Cumulative energy consumption during the lifespan. Source: e2stormed

6.1.3 Cumulative emissions

The comparative on cumulative emissions of each proposition are presented in the graph below:

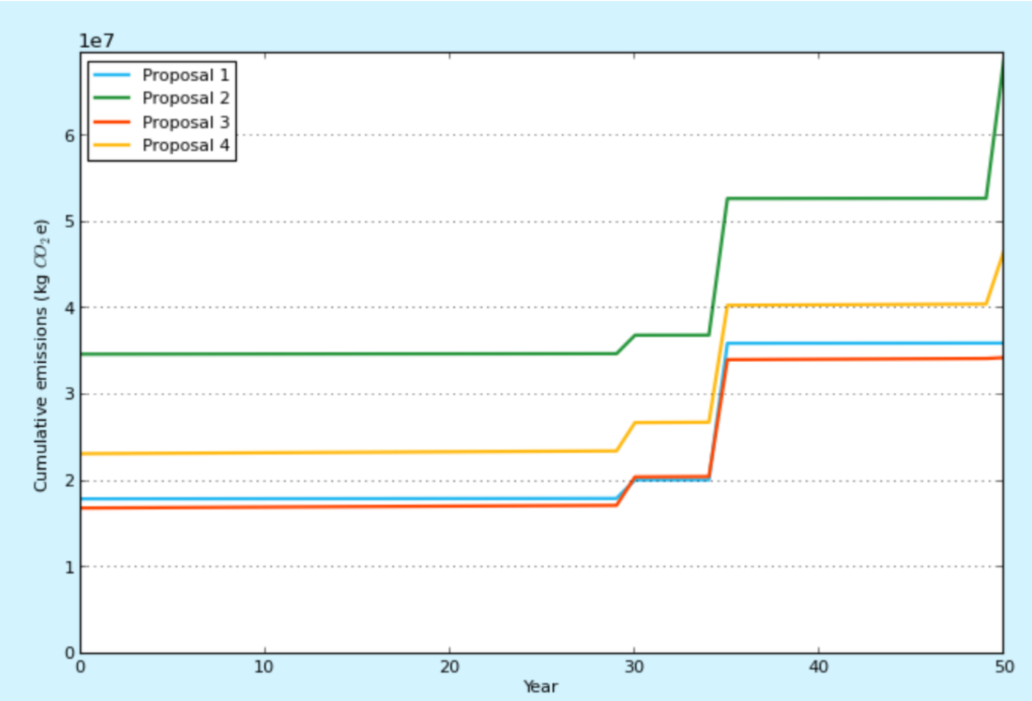

Figure 11. Cumulative emissions during the lifespan. Source: e2stormed

The emissions of the proposals are presented in accordance with those of energy consumption, as energy consumption is one of the largest contributors to CO2 equivalent emissions. In the proposed scenarios, in no case do emissions decrease over time.

6.2 Multi-criteria analyse

The E2stormed programme offers the possibility to choose the quantitative and qualitative criteria to be used for decision making. The programme is not intended to make the decision itself, but to guide decision-makers to make the most appropriate choice. The steps for the multi-criteria analysis are identify criteria, scoring by establishing the performance of each scenario against criterion, apply weighting and ranking of scenarios.

The steps for the multi-criterial análisis are:

- Identify criterion
- Scoring: establish the performance of each scenario against criterion
- Apply weighting
- Ranking of scenarios

The software offers a variety of criteria for the user to choose the most suitable ones by grouping them into financial, energy, emissions, environmental and water quality and other quantitative criteria.

- **Financial criteria:**
  - Cost of stormwater management
  - Net cost of storm water management
  - Construction and land take cost
  - Maintenance cost
  - Total construction and maintenance cost
  - Water reuse net benefits
  - Stormwater treatment and conveyance cost
  - Building insulation benefits
  - Flood protection benefits

- **Energy criteria**
  - Energy consumed by stormwater management
  - Net energy consumed by stormwater management
  - Energy consumed during construction
  - Energy consumed in maintenance
  - Energy consumed in construction and maintenance
  - Water reuse net energy saved
  - Treatment and conveyance energy consumption
  - Building insulation energy saved

- **Emissions criteria**
  - Emissions due to stormwater management
  - Net emissions due to stormwater management
  - Emissions during construction
  - Emissions in maintenance
  - Total emissions during construction and maintenance
  - Water reuse net emissions avoided
  - Treatment and conveyance emissions
  - Building insulation emissions avoided
  - Carbon dioxide reduced by vegetation

- **Other quantitative criteria**
  - Volume of water reused
  - Volume of runoff produced
  - Volume of discharge from Combined Sewer Overflows
  - Number of CSO spills per year
  - Peak outflow rate
  - Aquifer recharge and evapotranspiration
  - Water losses in the network

- **Environmental and water quality criteria**
  - Global outflow water quality
  - Suspended solids removal efficiency
  - Nutrients removal efficiency
  - Heavy metals removal efficiency
  - Evaluation of ecosystem services

Six criteria have been chosen for the multi-criteria analysis of the project:

1. Net cost of stormwater management
2. Peak outflow rate
3. Global outflow water quality
4. Evaluation of ecosystem services
5. Net energy consumed by stormwater management
6. Net emissions of stormwater management

6.2.1 Criteria weight

To obtain the global score of each scenario according to the multi-criteria analysis, a weight must be defined for each criterion. A higher weight indicates that the criterion will be more important in the decision making. The sum of the weight of all the criteria should be equal to 100%. Therefore, the global score for each scenario is obtained with the following formula:

$$GS_i = \sum_{j=1}^{n} W_j * UV_{ij}$$

Where :

- GSi (adimensional) is the global scorer of scenario i
- N (adimensional) is the number of criteria
- Wj (adimensional) is the weight of the criterion j
- UVij (adimensional) is the utility value of the scenario i in the criterion j

The weights chosen for each of the criteria are as follows:

Table 24. Criteria weight

Criteria		Weight
Net cost of stormwater management	C1	40%
Peak outflow rate	C2	35%
Global outflow water quality	C3	7%
Evaluation of ecosystem services	C4	7%
Net energy consumed by stormwater management	C5	5,5%
Net emissions of stormwater management	C6	5,5%

It is important to bear in mind how crucial this step is for the choice of the optimal solution, as a variation in the weight of a criterion could change the order of preference of alternatives.

In the case of the study, we have aimed to give greater weight to the net cost of stormwater management and the peak outflow rate, since in practice cost is one of the major limiting factors in the choice of solutions and the peak outflow rate is one of the most important criteria in the choice of solutions. It has already been mentioned throughout the project that the main objective is to reduce the discharge flow rate, which is why it has been considered that it should have a high weight.

6.2.2 Extreme utility values

The results of the analysis also depend on which values are used to give a utility of 100% or 0% for each criterion. For this purpose, a minimum value of 0 has been taken for all criteria and the maximum value for all propositions has been taken as the highest value.

Table 25. Best and worst value for each criteria

Criteria		Best value (utility 100%)	Worst value (utility 0%)
Net cost of stormwater management (€)	C1	0	7.579 e07
Peak outflow rate (m3/s)	C2	0	33.29
Global outflow water quality	C3	Very high	Very low
Evaluation of ecosystem services	C4	Very high	None
Net energy consumed by stormwater management (kWh)	C5	0	2.202 e08
Net emissions of stormwater management (kg CO2e)	C6	0	6.9517 e07

6.2.3 Results

In this way, the E2stormed programme provides both numerical and visual results in the form of a diagram that reads as follows:

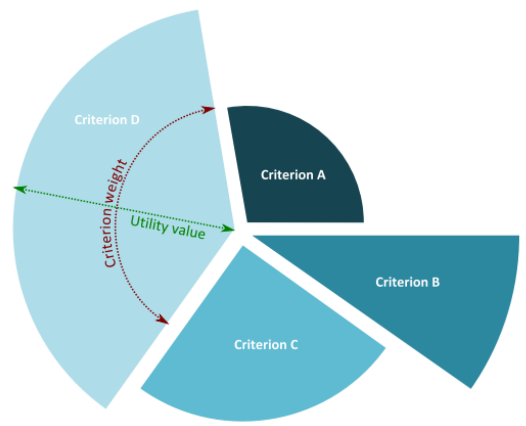

Figure 12. Diagram of result of multi-criteria analysis per scenario

For each scenario, we obtain the resulting diagram:

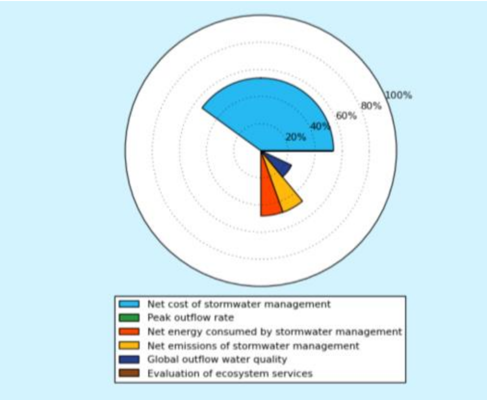

Figure 13. Diagram of result multi-criteria analysis. Alternative 1. Source: E2stormed

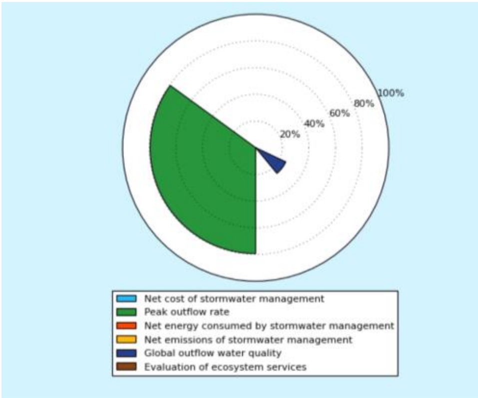

Figure 14. Diagram of result multi-criteria analysis. Alternative 2. Source: E2stormed

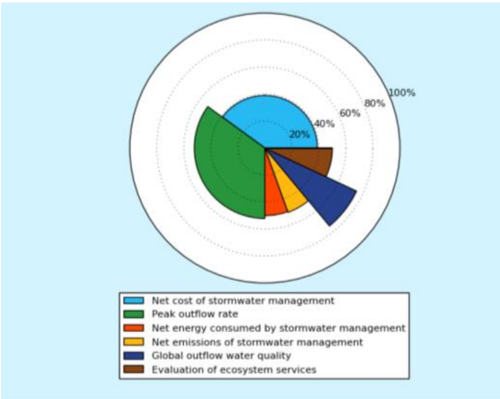

Figure 15. Diagram of result multi-criteria analysis. Alternative 3. Source: E2stormed

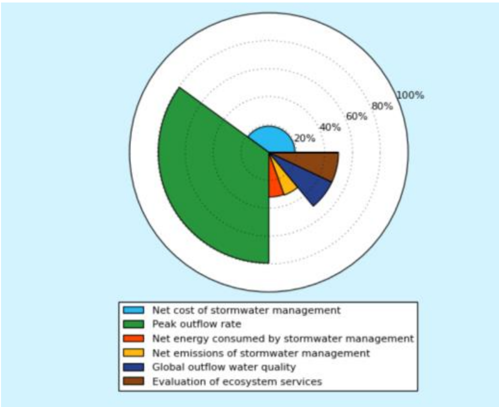

Figure 16. Diagram of result multi-criteria analysis. Alternative 4. Source: E2stormed

Finally, from these results we obtain the overall score for each alternative to compare all scenarios.

Table 26. Global results of multi-criteria analysis

Criteria	Proposal 1	Proposal 2	Proposal 3	Proposal 4
Net cost of stormwater management (%)	21,44	0,00	15,55	7,49
Peak outflow rate (%)	0,00	27,77	18,27	27,77
Global outflow water quality (%)	1,75	1,75	5,25	3,5
Evaluation of ecosystem services (%)	0,00	0,00	3,5	3,5
Net energy consumed by stormwater management (%)	2,64	0,00	2,76	1,76
Net emissions of stormwater management (%)	2,65	0,00	2,78	1,79
TOTAL (%)	**28,49**	**29,52**	**48,12**	**45,81**

It is represented graphically by a histogram that identifies the results of each criterion applied to each alternative, as shown in the following image:

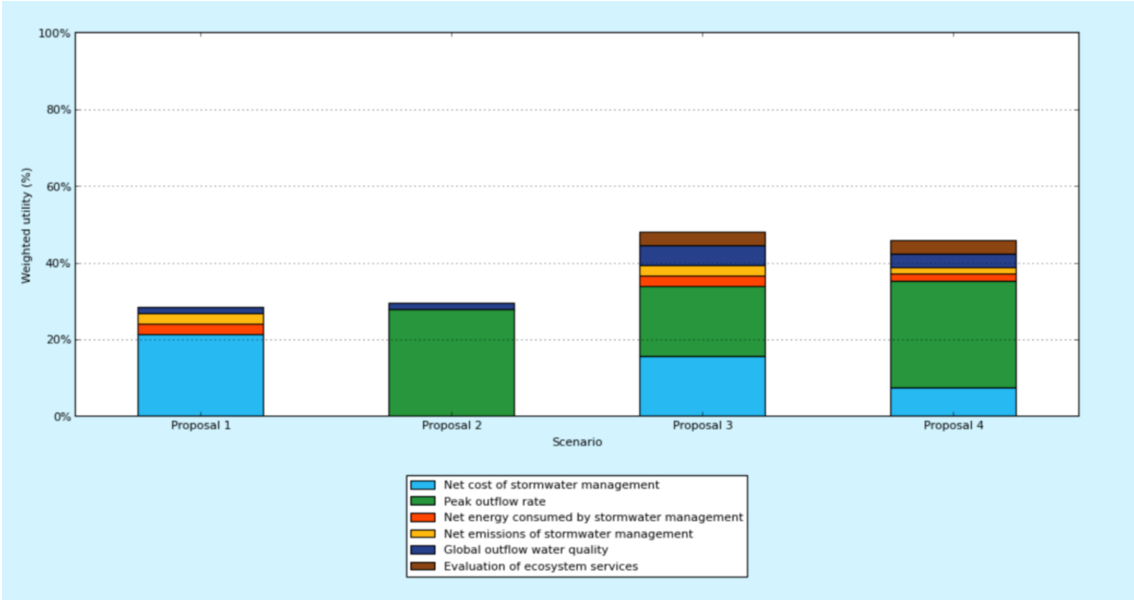

Figure 17. Histogram of global results of multi-criteria analysis

After the multi-criteria analysis it is concluded that **the optimal solution is alternative 3**, where the sustainable drainage systems are implemented but it does not include a storm tank and therefore does not reach the target peak flow in the natural state.

However, alternative 4 is located very close to it. Analysing in detail the reasons why alternative 3 is located in a higher position than alternative 4, the following has been concluded:

Alternative 3 ranks first due to the energy, emissions and environmental benefits of its drainage system, although in the combined hydraulics and finance aspects it is inferior to Alternative 4.

Table 27. Comparison on results of alternative 3 and 4

Criteria	Proposal 3	Proposal 4
Financial and hydraulic criteria (%) C1 + C2	33.82	35.26
Environmental and energetic criteria (%) C3 + C4 + C5 + C6	14.3	10.55
TOTAL (%)	**48,12**	**45,81**

This indicates that, for our analysis, the benefit that the construction of a storm tank would bring in terms of discharge flow makes it the best option even considering its costs. However, the added implementation of the tank and its pumping system, with its consequences in terms of energy and environmental criteria, makes that this alternative falls into second position. Finally, it is interpreted that the proximity of proposals 3 and 4 suggests that the prioritization of the alternatives is considerably sensitive to the weights given to the criteria

6.2.4 Analysis sensitivity

The proximity of proposals 3 and 4 suggests that the prioritisation of the alternatives is considerably sensitive to the weights given to the criteria. For information purposes, two different cases in which the weights of the criteria have been varied are shown below.

In the first case, greater weight has been given to the hydraulic and financial criteria, leaving 10% for the rest of the criteria. With this analysis we would obtain that the optimal solution is alternative 4.

Table 28. First case of sensitive analysis weight

Criteria		Weight
Net cost of stormwater management	C1	45%
Peak outflow rate	C2	45%
Global outflow water quality	C3	2.5%
Evaluation of ecosystem services	C4	2.5%
Net energy consumed by stormwater management	C5	2.5%
Net emissions of stormwater management	C6	2.5%

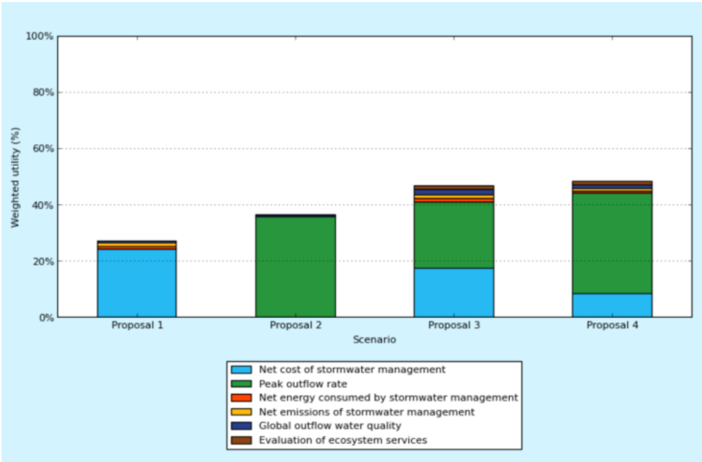

Figure 18. Histogram of global results of multi-criteria analysis in first case of sensitive analysis

In the second case, environmental and energy criteria have been removed, simulating a situation at the end of the last century, when these criteria were not considered and priority was given to cost and result. Following this casuistry, the optimal option is alternative 4 and it is worth noting that alternative 2 is close to alternative 3.

Table 29. Second case of sensitive analysis weight

Criteria		Weight
Net cost of stormwater management	C1	50%
Peak outflow rate	C2	50%
Global outflow water quality	C3	0%
Evaluation of ecosystem services	C4	0%
Net energy consumed by stormwater management	C5	0%
Net emissions of stormwater management	C6	0%

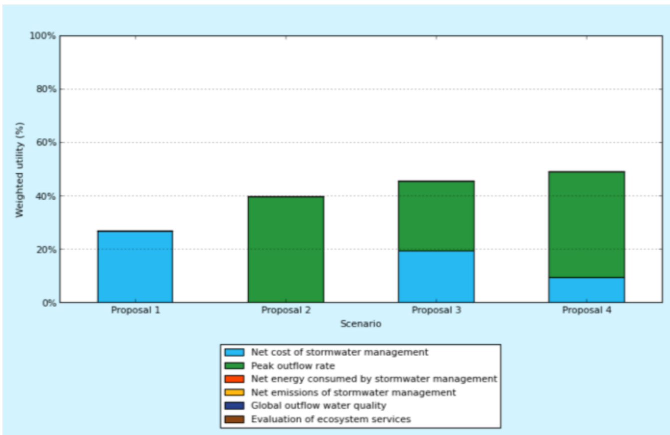

Figure 19. Histogram of global results of multi-criteria analysis in second case of sensitive analysis

This comparison serves as an example to show that the e2stormed programme is a decision support programme, but that it is never the decision-maker itself. The project is very sensitive to the distribution of weights and these weights, being subjective, could cause the solution to tilt towards one alternative or another.

For the development of the project, we will continue with the results obtained in the initial casuistry, as it is considered that the weights chosen are those that most faithfully reproduce the objectives pursued in this study, **identifying alternative 3 as the optimal one**.

ANNEX Nº5: DEVELOPMENT OF THE OPTIMAL SOLUTION

SUSTAINABLE DRAINAGE SYSTEMS (SUDS) IN INDUSTRIAL AREAS:
APPLICATION TO THE INDUSTRIAL AREA IN QUART DE POBLET (VALENCIA)

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Summary

TABLES INDEX

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1 OBJECT

This annex will develop the description of the chosen solution of the drainage system for the Quart de Poblet industrial estate to understand it in detail.

Throughout the study, the different drainage system options have been listed and emphasis has been placed on sustainable drainage techniques, thus supporting new techniques for the achievement of the UN Sustainable Development Goals. After the comparative analysis it has been shown that alternative 3, related to the implementation of sustainable drainage systems, is the most suitable alternative in terms of financial, hydraulic, energy, social and environmental criteria Alternative 3 represents all the benefits of implementing SUDS: controlling the quantity of runoff, managing the quality of the runoff to prevent pollution and creating and sustaining better places for people, by the amenity, and nature, by the biodiversity.

A more in-depth detail, compared to the presentation of alternatives, of the adopted solution in terms of its transport infrastructures, its registration systems and its sustainable drainage elements is presented below.

2 TRANSPORT INFRASTRUCTURE. COLLECTORS

The runoff conveyance infrastructure consists of a network of collectors using pipes of two different materials depending on the size.

The pipes with a diameter of 1200 mm or less will be made of corrugated PVC material. The diameters chosen were obtained from a SANECOR manufacturer's catalogue.

Table 1. PVC pipe's diameters. Source: SANECOR

DN (mm)	Ø Exterior (mm)	Ø Interior (mm)
400	400	364
500	500	452
630	630	590
800	800	775
1000	1000	970
1200	1200	1103

The manufacturer proposes pipes of stiffness ≥ 8 KN/m2 with nominal diameters between 160 and 1200 mm. As indicated in the Regulations for sewerage and urban drainage works in the city of Valencia, only diameters greater than 335 mm will be considered to avoid obstructions and facilitate cleaning work.

Figure 1. PVC pipe for diameters of less than 1200 mm. Source: SANECOR

In pipes where diameters larger than 1200 mm are required, RIBLOC has been chosen as the reference manufacturer. The CONCRETLOC series is composed of rigid PVC pipes, helically shaped with a profile that presents a wall structured by means of "T" shaped stiffeners and reinforcement with galvanized steel profile, providing the pipe with a high circumferential rigidity.

Table 2. PVC with stiffeners pipe's diameters. Source: RIBLOC

DN (mm)	Ø Exterior (mm)	Ø Interior (mm)
1300	1300	1268
1400	1400	1368
1500	1500	1468
1600	1600	1568
1700	1700	1668
1800	1800	1768
1900	1900	1868
2000	2000	1968
2100	2100	2068
2200	2200	2168
2300	2300	2268
2400	2400	2368
2500	2500	2468
2600	2600	2568
2700	2700	2668
2800	2800	2768
2900	2900	2868
3000	3000	2968

Figure 2. PVC pipe for diameters of more than 1200 mm. Source: RIBLOC

The collectors will be installed by introducing the pipes into a trench excavation and then pouring concrete to protect the pipe and prevent it from moving from its location. In addition, the concrete will be reinforced in the upper part of the pipe to resist the tensile stresses that could be caused.

The typical section of the collector trench is shown in the following image:

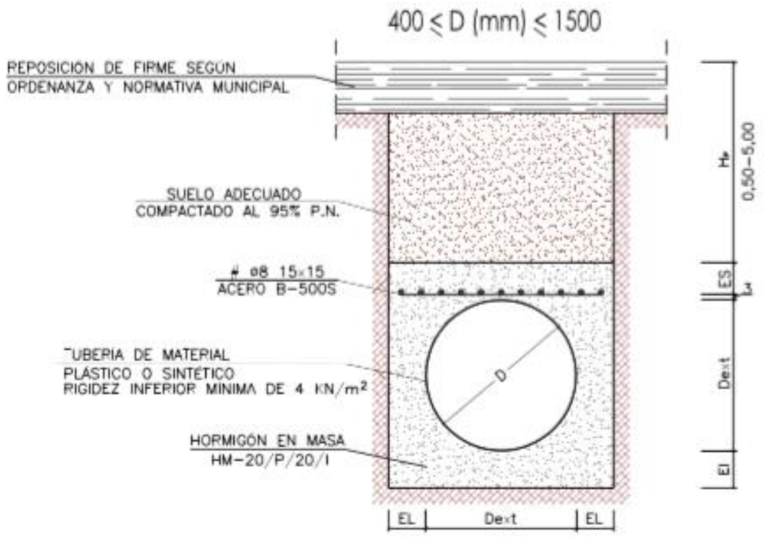

Figure 3. Collector trench type. Source: Ayuntamiento de Valencia, 2015

The values of minimum and maximum depth and cover will depend on the diameter of the pipe, the following being those included in the Valencian regulations:

DIÁMETRO APROXIMADO		ES (cm)	EI (cm)	EL (cm)
Ext (mm)	Int (mm)			
400	386	15	15	15
450	436	15	15	15
500	486	15	15	15
600	580	15	15	15
700	680	15	15	15
800	780	15	15	15
900	876	15	15	15
1000	976	15	15	15
1100	1076	20	20	20
1200	1176	20	20	20
1300	1268	20	20	20
1400	1368	20	20	25
1500	1468	20	20	25

Figure 4. Trench excavation dimensions values. Source : Ayuntamiento de Valencia, 2015

The collectors have been designed in such a way that these maximum and minimum values are respected.

In the case of collectors with a diameter greater than 1200 mm, whose values are not included in the regulations, the technical data sheet provided by the manufacturer and its layout recommendations shall be used.

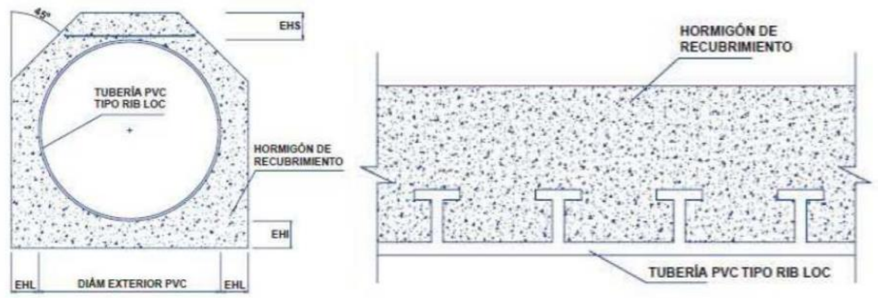

Figure 5. Scheme type of collectors disposition for diameters of more than 1200 mm. Source: RIBLOC

The solution adopted offers the following quantities in terms of pipe diameters. Generating a total of **10,725 meters** of collector network length.

Table 3. Collectors' pipes and its length

DN (mm)	Length (m)
400	600,3
500	396,5
630	896,2
800	1761,5
1000	1930,7
1200	1048,5
1300	1104,0
1400	92,9
1500	279,7
1600	0,0
1700	172,3
1800	387,8
1900	193,7
2000	338,8
2100	0,0
2200	188,6
2300	181,3
2400	220,7
2500	144,4
2600	222,8
2700	0,0
2800	0,0
2900	263,0
3000	301,5
TOTAL	**10725,2**

3 REGISTRATION SYSTEMS

The registration systems identified are the manholes, the registration wells in case the dimensions pass a threshold for the manholes, the service connections and the scuppers.

3.1 Manholes

The manholes shall be placed at the following points:

- Beginning of each section
- Change of direction, diameter or slope of mains
- Change of network section
- Incorporation of other collectors
- Connection to service connections and scuppers

In addition, the maximum distance between manholes has been limited to 100 meters along each section to facilitate maintenance, inspection and cleaning.

In accordance with the Regulations for sewerage and urban drainage works in the city of Valencia (Ayuntamiento de Valencia, 2015), manholes with diameters of between 400 and 1500 mm will be installed for collectors with diameters of between 1000 and 1500 mm. The following dimensions will be used for the design of the manholes:

Table 4. Manhole diameter by collector diameter

⌀ Collector (mm)	⌀ Manhole (mm)
400	1000
500	1000
630	1000
800	1000
1000	1000
1200	1200
1300	1500
1400	1500
1500	1500

In cases where the collectors are located at depths equal to or less than 1.2 m, the manholes may be made of 1 ft. honeycomb or perforated brick, internally covered with M-700 water-repellent cement, burnished.

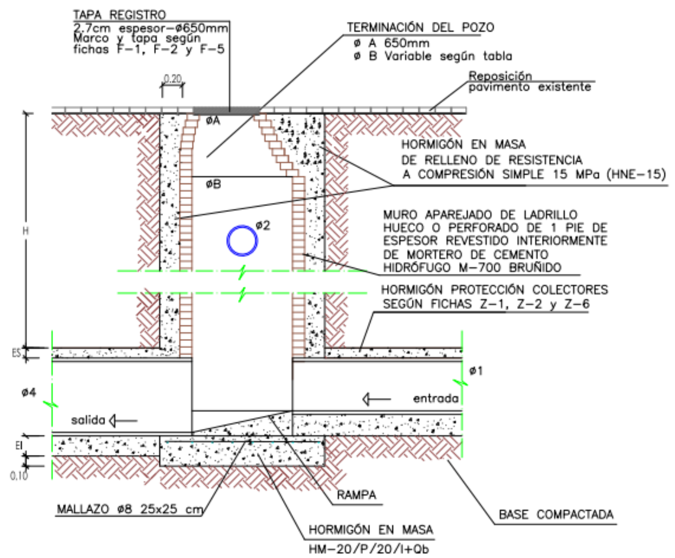

Figure 6. Brickwork manhole type elevation. Source: Ayuntamiento de Valencia, 2015

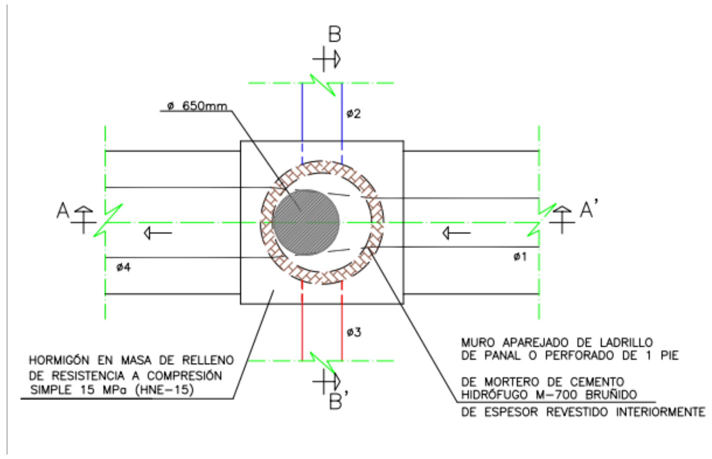

Figure 7. Brickwork manhole type plant. Source: Ayuntamiento de Valencia, 2015

For all other depths, the manholes will be made of prefabricated elements.

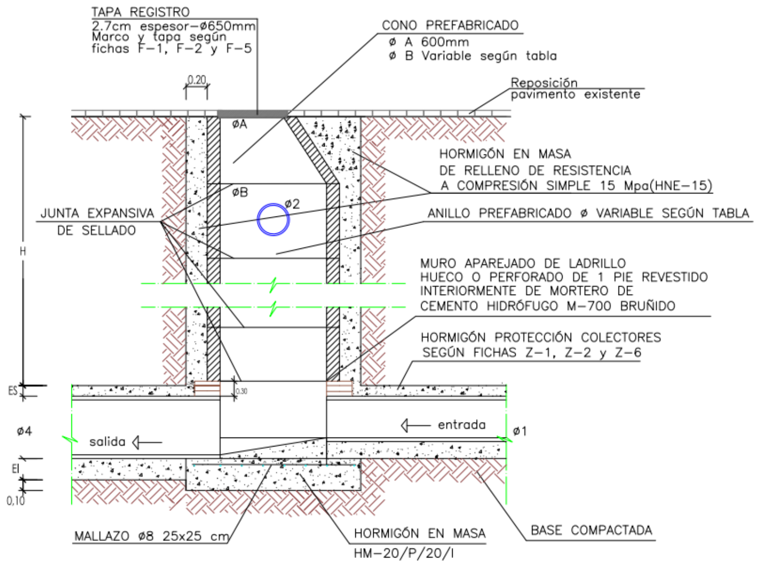

Figure 8. Prefabricated manhole type elevation. Source: Ayuntamiento de Valencia, 2015

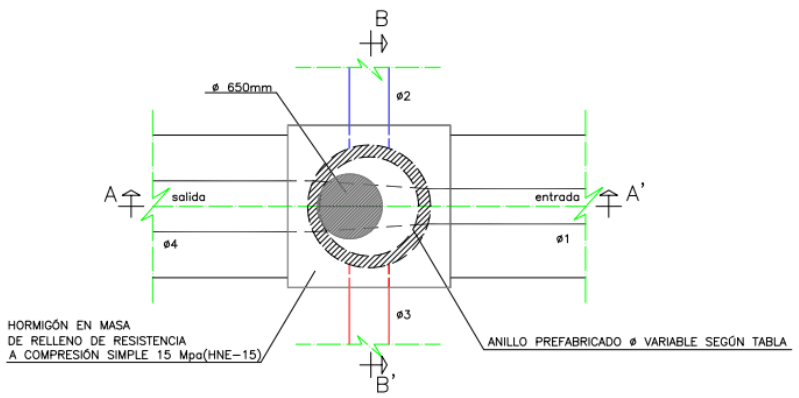

Figure 9. Prefabricated manhole type plant. Source: Ayuntamiento de Valencia, 2015

Provided that the height from the base of the manhole to the ground level is greater than 0.7 m, the elevation of the manholes will be provided with different heightening modules, which correspond to a circular section open at both ends.

Considering these determinations, the study area has a total of **106 manholes:**

Table 5. Manholes

Catchment	Manholes
N1	21
N2	20
N3	65
TOTAL	**106**

3.2 Registration wells

In the case of collectors larger than 1500 mm in diameter, registration wells will be installed instead of manholes to be able to connect the different collectors. Their installation points are the same as for manholes:

- Beginning of each section
- Change of direction, diameter or slope of the network
- Change of network section
- Incorporation of other collectors
- Connection with service connections and scuppers
- Maximum distance of 100 metres

The registration wells shall be made up of a combination of different elements or modules joined together by overlapping, and intercalating expansive sealing joints, which make these joints sufficiently watertight.

The materials shall be of "in situ" base with prefabricated concrete elevation. The base shall consist of clean concrete with a minimum compressive strength of 15 MPa, a reinforced concrete transition and/or closure slab and filling of the registration wall with mass concrete.

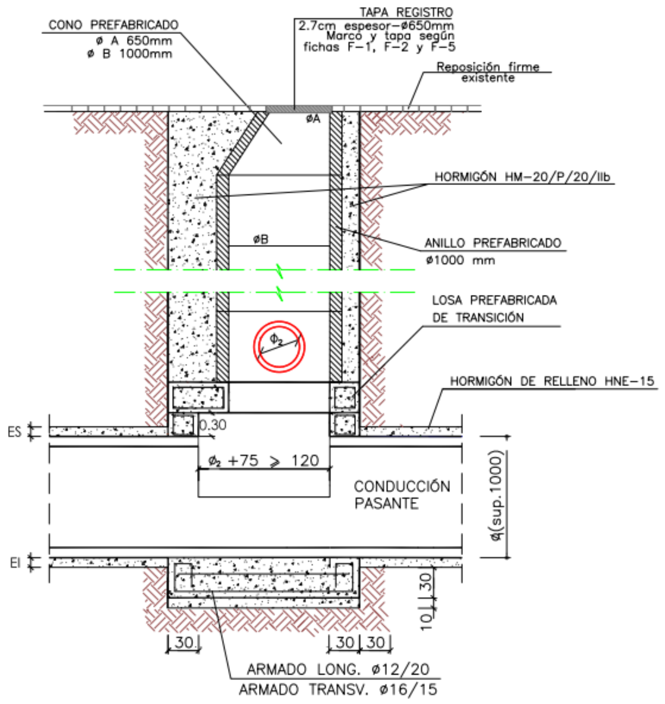

Figure 10. Registration well type elevation. Source: Ayuntamiento de Valencia, 2015

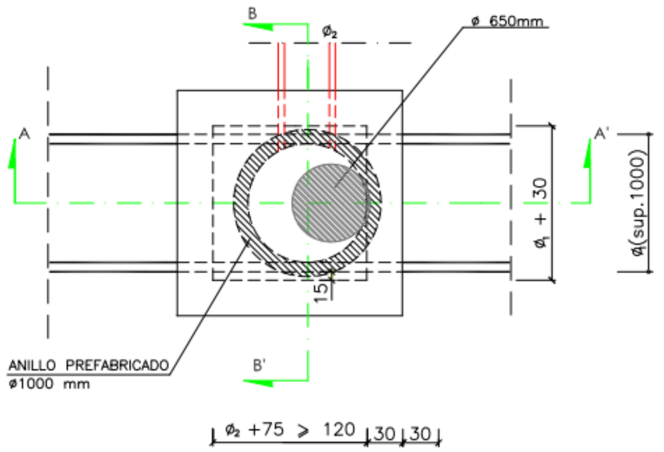

Figure 11. Registration well type plant. Source: Ayuntamiento de Valencia, 2015

Considering these determinations, the study area has a total of 32 **registration wells**:

Table 6. Registration wells

Catchment	Registration wells
N1	0
N2	11
N3	21
TOTAL	**32**

3.3 Service connections

As indicated before, the design of the collector network for the private subcatchments is not the subject of this study, but it is important to consider the stormwater from these plots in the public collector network, as the amount of water from private subcatchments is a high percentage of the total catchment.

In this case, the use of the service connections will be made to join the rainwater from the private subcatchments to the network of collectors in the street subcatchments. A diagram of this service connection can be seen below.

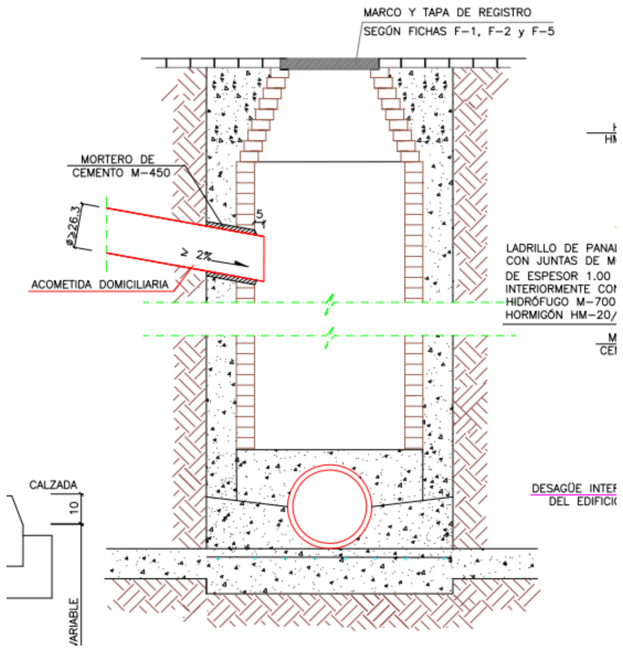

Figure 12. Service connection type elevation. Source: Ayuntamiento de Valencia, 2015

3.4 Scuppers

Scuppers are the elements that collect runoff water for introduction into the collector network.

The location of the scuppers is where the most efficient interception of rainwater is possible.

- At the kerb: on roads with a cross slope towards the pavements.
- In the centre: on roads sloping towards the road axis.

They are to be implemented with continuous ramps with a minimum cross slope of 10% to convey surface runoff to the scuppers.

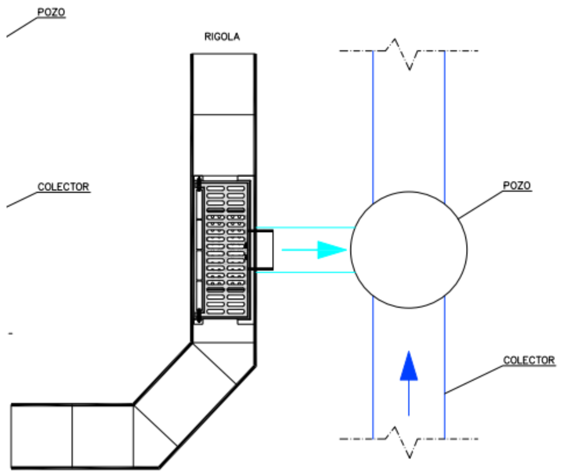

Figure 13. Scuppers scheme. Source: Ayuntamiento de Valencia, 2015

4 SUSTAINABLE DRAINAGE SYSTEM

The chosen solution seeks the implementation of sustainable drainage systems. In the proposal of solutions, the reasons why the typologies were chosen were discussed. In this section the structural sections for each of the sustainable drainage techniques will be presented.

4.1 Pervious pavements

The pervious pavements have been laid on one or two side bands of the streets whose width allowed it. As indicated, the width of the pavements will be 3 meters to comply with the minimum parking width indicated in the Law of Territorial, Urban and Landscape Planning of the Comunidad Valenciana (LOTUP).

In this way, the width of the street will indicate the possibility of having a lateral parking band with pervious pavement, two lateral parking bands or, if the width is sufficiently wide, adding a central median that functions as a rain garden. The following is the proposed road division depending on the total width.

Table 7. Cross-section type distribution for pervious pavements

Total width (m)	Impermeable área width (m)	Pervious pavement width (m)	Central median rain garden width (m)
16	13	3(1)	0
18	15	3(1)	0
20	14	3(1)	3
22	16	3(1)	3
25	16	6(2)	3
30	18	6(2)	6

(1) Arranged in one side band
(2) Arranged in two lateral bands

The cross-section of the permeable pavements to be implemented in the solution has been designed based on literature references (Lafarge, 2013), (J. Rodriguez-Hernadez et al., 2011) , (Rodríguez Hernández, J., 2.008). (Woods-Ballard et al., 2015), (Sañudo-Fontaneda et al., 2014), (Beeldens and Herrier, 2006) and (Castro, J. 2009).

The pervious pavement section shall be composed of the following layers:

- Superficial porous concrete layer
- Sub-superficial gravel layer 4/8
- Geotextile
- Sub-base gravel layer 20/40
- Drain
- Geomembrane

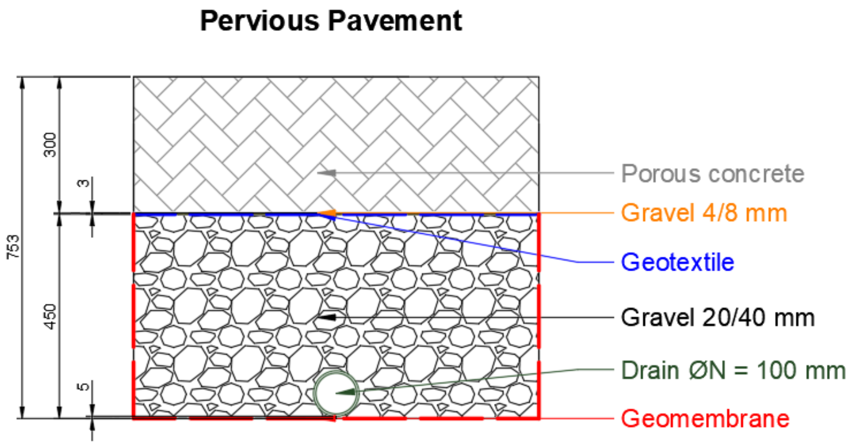

Figure 14. Cross-section on pervious pavements

4.1.1 Superficial porous concrete layer

The porous concrete surface layer shall be the first top layer of the section that will be in contact with the vehicles. This layer shall have the following characteristics:

- The thickness of the layer shall be 300 mm.
- The void ratio for the porous mix shall correspond to 25%.
- Its stiffness shall be between 25 and 45 GPa, with a common value of 38 GPa.
- The specific weight of the porous concrete shall be between 1700 and 2000 kg/m3.
- A particle size without fines or at least with a low content shall be required. Identifying a minimum aggregate size of 5mm. The maximum aggregate size shall be 20 mm.
- The binder used shall be hydraulic cement, with a content of 267-326 kg/m3 and a water-cement ratio of 0.26-0.38. The cement shall be modified with silica fume admixtures, superplasticiser and organic polymers to move from average compressive strengths of 20 MPa to ensure a compressive strength more than 45 MPa.
- The minimum flexural tensile strength shall be 2.5 MPa.
- The permeability of the porous concrete shall be around 250 mm/h.

4.1.2 Sub-superficial gravel layer 4/8

A sub-layer of gravel shall be placed under the porous concrete surface layer with a thickness of 30 mm and a minimum aggregate size of 4 mm and a maximum of 8 mm, with the aggregate size increasing with depth.

4.1.3 Geotectile sheet

A non-woven geotextile sheet will be placed between the surface layer and the storage layer, which will act as a filter and separation to prevent the transfer of fines and the loss of support at the base. This sheet will also retain most of the pollutants carried by the filtered water. Its characteristics will be:

- Mass per unit surface area between 125 and 160 gr/m2.
- Vertical permeability between 100 and 130 mm/s.

4.1.4 Sub-base gravel layer 20/40

The storage layer of gravel shall consist of the following specifications:

- The thickness shall be 450 mm.
- The aggregates shall be artificial and from quarries.
- The granulometry shall be as uniform as possible, trying to eliminate fines smaller than 2 mm.
- The voids index shall be 50%.
- The Los Angeles coefficient shall be less than 30.
- The sand equivalent shall be higher than 40.
- The particle size shall be between 20 and 40 mm.

4.1.5 Geomembrane sheet

Finally, a layer of geomembrane shall be laid to impermeabilize the entire structure and thus prevent infiltration. This layer must ensure a total sealing of the esplanade that will allow for a waterproof structure that will function as a storage tank for the filtered water.

4.1.6 Drain pipe

The outlet of the stored water will be by means of a PVC drainage pipe buried in the gravels at 5 mm from the base. The nominal diameter of the pipe will be 100 mm and it will be lined with a geotextile to avoid obstructions in its grooves.

4.2 Rain gardens

The rain gardens have two types of sections. One with geomembrane and drainage and one without either, thus allowing infiltration.

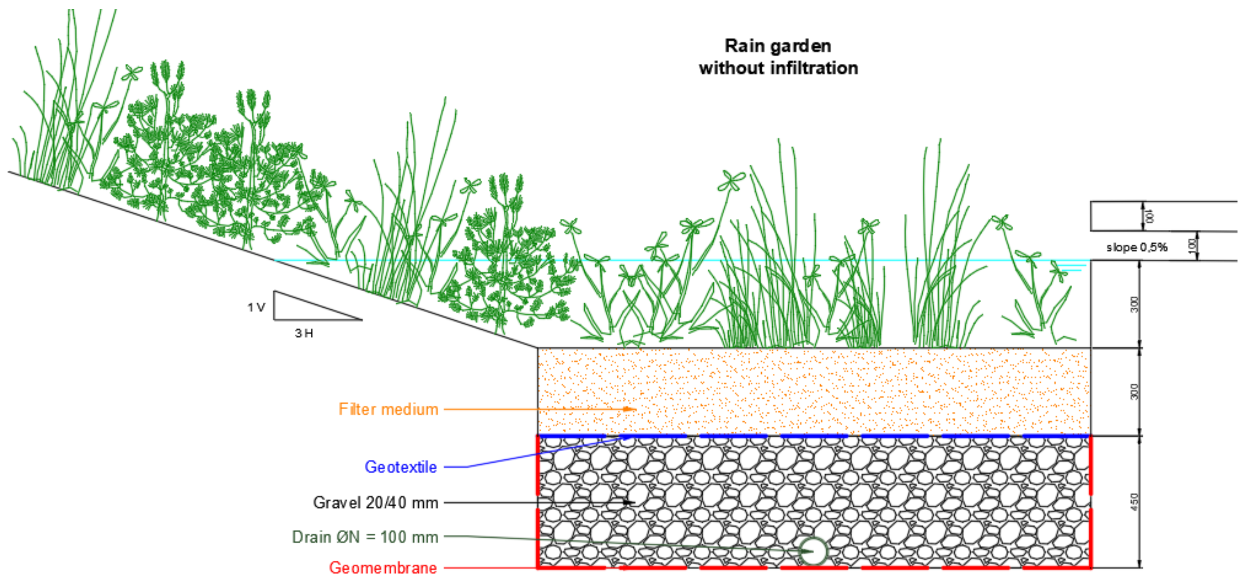

Figure 15. Cross-section on rain gardens without infiltration

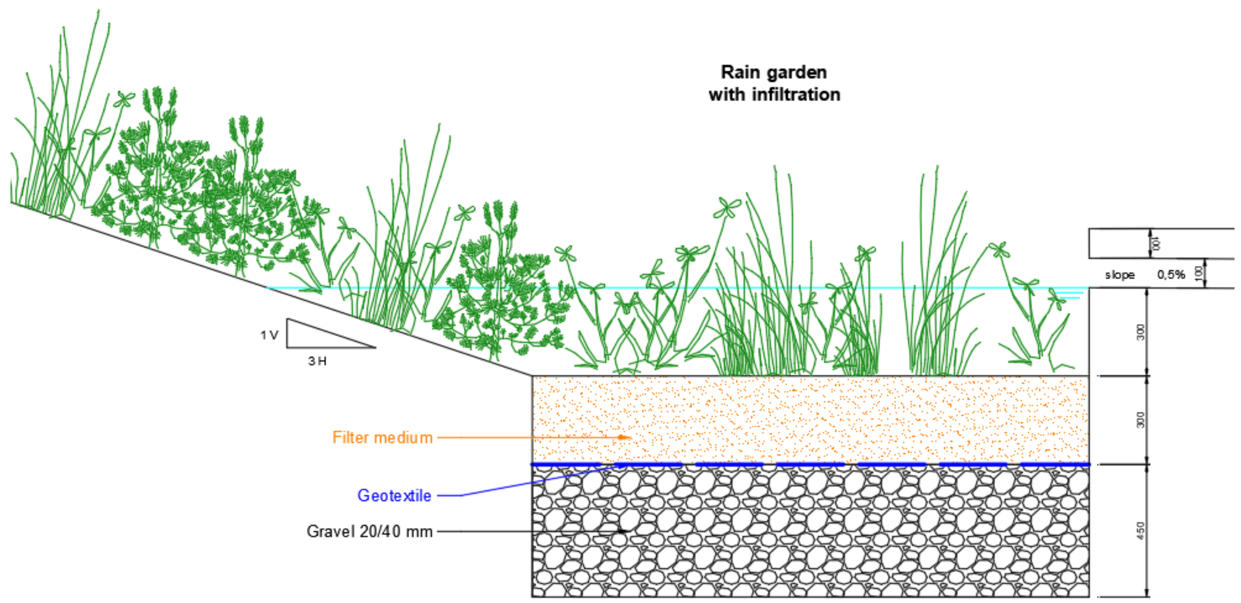

Figure 16. Cross-section on rain gardens with infiltration

The cross-sections shall consist of the following layers:

- Free surface
- Filter medium
- Geotextile
- Sub-base gravel layer 20/40
- Geomembrane (if no infiltration)
- Drain (if no infiltration)

4.2.1 Free Surface

The free surface area available in the rain gardens allows a water height of up to 300mm to be stored as a storage tank.

For events over the design height, a relief structure, normally a box spillway, with a height of 100mm is included to safely direct runoff downstream, with a slope of 0.5%. A buffer shall also be provided between the top of the spillway and the finished elevation of the adjacent 100 mm surface.

The free surface shall be covered with native vegetation, with a minimum density of 6 pcs/m2 and a minimum vegetation volume fraction of 0.1/m3.

4.2.2 Filter medium

The filter medium has the following characteristics:
- The thickness shall be 300 mm.
- The permeability shall be 250 mm/h
- The porosity will be 0.5
- The organic matter content will be between 3-5%.
- The pH will be between 5.5 and 8.5
- Electrical conductivity will be less than 3300 microS/cm

4.2.3 Geotextile sheet

A non-woven geotextile sheet is placed between the filter medium and the storage layer to ensure that soil particles do not pass from one layer to the other. This sheet has the same functions as in pervious pavements and the same characteristics:
- Mass per unit area between 125 and 160 gr/m2.
- Vertical permeability between 100 and 130 mm/s.

4.2.4 Sub-base gravel layer 20/40

The gravel sub-base shall be more permeable than the filter medium.
- The thickness shall be 450 mm
- Aggregate size shall be between 20 and 40 mm.
- The void ratio shall be 50%.
- The aggregates shall be artificial and from quarries.

4.2.5 Geomembrane sheet

In the event that the rain garden does not allow infiltration, i.e. when they are at the head of the pollutant mitigation chain, a geomembrane sheet will be provided to waterproof the structure.

4.2.6 Drain pipe

If no infiltration is allowed, the stored water will be drained by means of a PVC drainage pipe buried in the gravel 5 mm from the base. The nominal diameter of the pipe shall be 100 mm and it shall be lined with a geotextile to avoid obstructions in its grooves.

4.3 Infiltration basins

Infiltration basins have a similar cross-section to rain gardens that allow infiltration, except that these basins have a free surface area that allows for greater water infiltration and do not have a bioretention function, as SuDS bioretention elements cannot be larger than 0.8 ha in size.

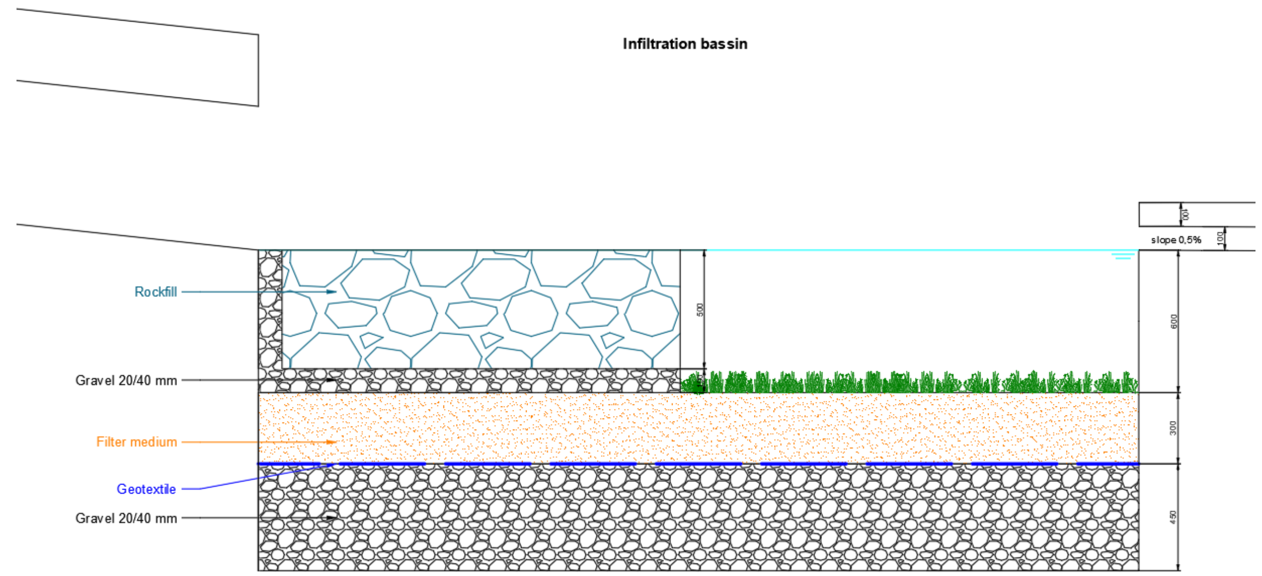

Figure 17. Cross-section of infiltration basin

The cross-section shall consist of the following layers:
- Free surface
- Rockfill layer
- Subbase gravel 4/8 layer under rockfill
- Filter medium
- Geotextile
- Subbase gravel 20/40 layer

4.3.1 Free Surface

The free surface area available in the infiltration areas allows for the storage of up to 600 mm of water as a reservoir, which is completely emptied within 48 hours, both at the base and at the sides. For this reason, the inlet infrastructure will be placed with the floor level at this height.

This infrastructure is a pipe whose diameter varies depending on whether we are in the infiltration basin of basin N1 or N2. As in the rain gardens, a relief structure is included, normally a box-type spillway, with a height of 100 mm to direct the runoff downstream in safe conditions, with a slope of 0.5% for events exceeding the design slope.

Moreover, a certain buffer between the upper level of the spillway and the finished level of the adjacent surface of 100 mm will also be proposed. In addition, the free surface shall be covered with native vegetation, with a minimum density of 16 pcs/m2.

4.3.2 Rockfill layer

The entrance shall have an energy dissipating element composed of a layer of rockfill with an average diameter of 250 mm. It shall be trapezoidal in shape and 50 mm thick.

4.3.3 Sub-base gravel 20/40 under rockfill

A layer of 100 mm thick gravel 20/40 mm shall be placed under the rockfill layer.

4.3.4 Filter medium

The filter medium has the following characteristics:

- The thickness shall be 300 mm.
- The permeability shall be 250 mm/h
- The porosity will be 0.5
- The organic matter content will be between 3-5%.
- The pH will be between 5.5 and 8.5
- Electrical conductivity will be less than 3300 microS/cm

4.3.5 Geotextile sheet

A non-woven geotextile sheet is placed between the filter medium and the storage layer to ensure that soil particles do not pass from one layer to the other. This sheet has the same functions as in pervious pavements and the same characteristics:

- Mass per unit area between 125 and 160 gr/m2.
- Vertical permeability between 100 and 130 mm/s.

4.3.6 Sub-base gravel layer 20/40

The gravel sub-base shall be more permeable than the filter medium.

- The thickness shall be 450 mm
- Aggregate size shall be between 20 and 40 mm.
- The void ratio shall be 50%.
- The aggregates shall be artificial and from quarries.

4.3.7 Geomembrane sheet

In the event that the rain garden does not allow infiltration, i.e. when they are at the head of the pollutant mitigation chain, a geomembrane sheet will be provided to waterproof the structure.

4.3.8 Drain pipe

If no infiltration is allowed, the stored water will be drained by means of a PVC drainage pipe buried in the gravel 5 mm from the base. The nominal diameter of the pipe shall be 100 mm and it shall be lined with a geotextile to avoid obstructions in its grooves.

ANNEX Nº6: MONITORING AND MAINTENANCE PROGRAM

SUSTAINABLE DRAINAGE SYSTEMS (SUDS) IN INDUSTRIAL AREAS:
APPLICATION TO THE INDUSTRIAL AREA IN QUART DE POBLET (VALENCIA)

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Summary

TABLES INDEX

FIGURES INDEX

1 OBJECT

This annex presents the monitoring and maintenance program for the solution for the implantation of scenario 3 with sustainable drainage systems.

The monitoring program will indicate all the elements and processes necessary to monitor the state of the water in terms of both quantity and quality. On the other hand, the maintenance program will be established, listing all the necessary tasks and their frequency for each SuDS element.

2 MONITORING PROGRAM

The monitoring program is developed to know the status of the infrastructure and to follow up on it. This program makes it possible to identify system malfunctions and prevent them before there are major consequences.

To this end, different monitoring elements are installed to track the quantity and quality of the water resource.

2.1 Quantity

The amount of runoff managed should be monitored at the most relevant points. Primarily the final discharge point, but there should also be a number of points scattered throughout the network to more easily identify the location of the problem.

At each of the control points, the hydraulic variables that can identify the amount of runoff managed and check whether the drainage system is working as planned, or whether there is a problem or a deficiency in the network, should be measured.

As is to be expected, most of the time the monitoring will give zero values because there will be no rainfall. Therefore, dataloggers will be implemented that will have the ability to vary the frequency of data collection depending on the amount of rainfall they detect, so that in dry periods data collection will be reduced. During the analysis, zero data will be eliminated and valid data will be sorted by precipitation events, considering the delimitation between one event and another if there is a period of 24 consecutive hours without rain.

2.1.1 Monitoring points

The points chosen for monitoring are:
- Infiltration basin of catchment N1
- Infiltration basin of catchment area N2
- Discharge point into La Saleta ravine

Figure 1. Monitoring points

The infiltration basins of the N1 and N2 catchments are interesting points to monitor because these SUDS are the end of the chain and have a great lamination power. The infiltration basins are the only elements that allow infiltration into the ground in these basins, so it is important to know the inflow and outflow, as well as the height of the layer of water.

It is important to know the inflow of the infiltration basins because in this way we can know the effectiveness of the performance of the different SUDS located upstream, such as permeable pavements and rain gardens.

On the other hand, by knowing the outflow and the height of the water layer we can know the efficiency of the infiltration of the infiltration basin by volume difference and it will also determine the efficiency of lamination of the infiltration basin.

The discharge point is the main monitoring point because it is the point where we know the flow and volume that is discharged into the receiving environment. Bearing in mind that one of the main objectives is to reduce the discharge flow, it is essential to know what the discharge flow is, as well as its volume. Furthermore, in view of the optimization of the SUDS it will be possible to know the number of events that discharge water into La Saleta ravine.

2.1.2 Quantity monitoring tools

2.1.2.1 Ultrasonic flowmeter

Located in the inlet collector of the infiltration basins, as well as in the discharge collector at the end point.

This instrument makes it possible to measure the speed and depth of the water flowing through the collector. Its position in the inlet collector to the infiltration pond allows to know the inlet flow to it and its position at the final discharge point allows to know the flow discharged to the receiving environment.

Figure 2. Ultrasonic flowmeter. Source: Mejoras energéticas

In terms of maintenance of the flowmeter, the flowmeter shall have an integrated battery to power the flowmeter, which shall be changed periodically. The life of this battery shall be considerable and provision shall be made for the battery to be sufficiently charged in case of expected storms.

2.1.2.2 Hydrostatic pressure sensor

The hydrostatic pressure sensor is used to determine the height of the water level inside the infiltration basins.

Figure 3. Hydrostatic pressure sensor. Source: Hyquest Solutions

To avoid possible deposits of material that could influence the measurements, the sensor shall be placed 1 cm above the base of the infiltration basin, taking this into account when interpreting the results.

The hydrostatic sensor also provides information on the outflow of the infiltration basin by means of a thin-walled triangular spillway. By measuring the height of the layer of water above the vertex of the spillway, this information can be used to obtain the flow rate from reference tables.

2.2 Quality

It has been decided that the quality monitoring points should be the same as the quantity monitoring points, since they are the most relevant for determining the proper functioning of the drainage system.

The process to be followed is as follows:

- A warning shall be sent after rainfall events exceeding a threshold.
- Following the warning, a responsible operator shall come to collect samples and replace instrumentation.
- Laboratory sampling shall be carried out within 24 hours.
- The results obtained shall be recorded in spreadsheets for analysis.

2.2.1 Quality monitoring instruments

Quality monitoring instruments are much less digital than quantity monitoring instruments. They consist of two bottles with a funnel which shall be protected with a wire mesh to prevent the entry of coarse particles.

The bottles will be placed at the points where it is necessary to know the quality, these being the inlet to the infiltration basins, the outlets of these and the final discharge point to know the quality of the water discharged into La Saleta ravine.

2.2.2 Parameters to be evaluated

The parameters to be measured to enable the evaluation of water quality shall be total suspended solids, biological and chemical oxygen demand, turbidity, total nitrogen and total phosphorus. The values of the above parameters shall comply with the different regulations governing the tests:

Table 1.Quality analysis parameters

Parameter	Análisis method	Range
Total Suspended Solids	UNE – EN 872	-
Biological Oxygen Demand after 5 days	Respirometry	0 – 80 mg O2/L
Chemical Oxygen Demand	ISO 15705	4 - 40 mg O2/L 10 - 150 mg O2/L 25 - 1500 mg O2/L
Turbidity	Turbidimetry	0,02 – 800 NTU
Total Nitrogen	ISO 11905 – 1	0,20 – 20 mg O2/L
Total Phosphorus	ISO 6878/1	0,01 – 5 mg P/L

3 MAINTENACE PROGRAM

The maintenance tasks of sustainable drainage systems focus on these four missions:

- Inspections to determine the level of efficiency and to plan maintenance requirements
- Operations and maintenance of the drainage system
- Landscape management

- Management of waste associated with contaminated soils or material as well as waste generated by maintenance.

All maintenance tasks should take into account the habitats found in the study area and the ecological aspects.

The most important requirements to be taken into consideration for maintenance are:
- Maintenance access: ensuring appropriate and permanent access to all points in the system where maintenance is required.
- Adequate pre-treatment systems to help trap sediment.
- Appropriate provision for temporary drainage while sediment management or other maintenance activities are carried out.
- Provision of storage areas for green waste such as cut vegetation or organic sediments

3.1 Maintenance manual

Ideally for the maintenance program there should be a maintenance manual which should be succinct and easy to use and which should include the following points:
- Location of all SUDS components in the study area.
- Summary of the SUDS design, how its components function, its purpose and potential for risk of effectiveness.
- Depth of silt to identify the need for maintenance.
- Visual indicators to identify the need for maintenance.
- Maintenance requirement.
- Explanation of proposed maintenance objectives and potential implications of not meeting these objectives.
- Identification of areas where certain activities are forbidden.
- Action plan for dealing with accidental releases of contaminants.
- Advice on what to do if excavation alterations are required by service companies and how this would affect SUDS.
- Contact details in the event that contamination is identified or the SUDS is not functioning correctly.

3.2 Maintenance level

The level of maintenance that will indicate the type of task to be carried out and the frequency with which it will be done depends on many aspects. Mainly the following have been identified:
- Characteristics of the SUDS components
- Size of the adjacent catchment from which the stormwater is collected in relation to the area of the SUDS
- The land use in adjacent catchments
- The level of urban development that is expected
- The SUDS planting scheme
- The habitat types that are intended to be created and how they are expected to evolve
- The amenity and visual impact requirements of the area.

According to these characteristics we will determine the task, the frequency, as well as the category among which we find:
- Periodic maintenance, including inspection
- Occasional maintenance
- Corrective maintenance

3.3 Maintenance proposal

Based on the above, the recommendations indicated in the main guides to sustainable drainage systems that have been used throughout this work (CIRIA, 2015), (Ayuntamiento de Madrid, 2018), (Ayuntamiento de Castellón, 2019) have been taken into account.

3.3.1 Pervious pavement

The maintenance proposal for permeable pavements is as follows:

Table 2. Pervious pavement maintenance proposal

Task	Type	Frequency	Element
Initial inspection after construction	Inspection	Monthly for the first three months	Pavement surface
Routine inspection for clogged areas or unwanted vegetation growth	Inspection	Six-monthly and after heavy rainfall	Pavement surface
Permeability testing to determine the need for corrective action	Inspection	Annually for the first three years	Pavement surface
Dry sweeping or standard vacuuming	Periodic	Six-monthly (spring and autumn)	Pavement surface
Remove unwanted vegetation on pavement and adjacent areas	Occasional	Annually or as needed	Pavement surface
Correction of vegetation or soil levels on adjacent surfaces that have risen up to 50 mm above pavement level	Corrective	Every 5 years or as needed	Pavement surface
Repairing any depressions, cracks or broken pavers that compromise the structural capacity of the pavement or present a risk to users and replacing joint material	Corrective	Every 5 years or as needed	Pavement surface
Rehabilitation of the surface and top of the sub-base by deep vacuuming, if permeability has been significantly reduced by clogging	Corrective	Every 15 years	Pavement surface
Reconstruction at the end of lifespan	Corrective	At end of lifespan (25 years)	Pavement surface

3.3.2 Rain garden

One of the most important aspects of rain gardens is related to vegetation. The proposed maintenance of rain gardens will be as follows:

Table 3. Rain gardens maintenance proposal

Task	Type	Frequency	Element
Remove leaves, trash, sediment and unwanted weeds from the infiltration surface	Periodic	Monthly	Surface, spillway and inlet
Remove sediment accumulated at the inlet and sediment that gets trapped in the spillway	Periodic	Annually or as needed	Surface, spillway and inlet
Irrigation to maintain good vegetation density	Periodic	Monthly	Vegetation
Replant sparsely vegetated areas	Corrective	When necessary	Vegetation
Regular checking for sediment accumulation, ponding, vegetation damage and erosion	Inspection	Six-monthly	Surface, spillway, inlet and vegetation
Technical inspection of inlet structures and infiltration surface (if applicable)	Inspection	Six-monthly and after heavy rains	Surface
Evaluation inspection of emptying time	Inspection	Six-monthly and after heavy rains	Surface
Filling of eroded areas and improvement of erosion protection if necessary	Corrective	When necessary	Surface, spillway, inlet and vegetation
Testing of permeability of the filter medium to determine the need for corrective action	Inspection	Biennially	Filter medium
Remove silt accumulations, scarify surface and backfill with substrate	Occasional	Every 3 years or as needed	Filter medium
Technical inspection for evidence of animal evidence in the interior	Inspection	Annually	Filter medium
Perform permeability test to determine need for corrective action	Inspection	Annually, for the first three years	Surface
Reconstruction at the end of lifespan	Corrective	At end of lifespan (30 years	Surface, vegetation, inlet, spillway, filter medium, gravels, drainage

3.3.3 Infiltration basin

The proposal for the infiltration basins is as follows:

Table 4. Infiltration basins maintenance proposal

Task	Type	Frequency	Element
Remove leaves, litter and unwanted weeds from the infiltration surface	Periodic	Monthly	Surface, spillway and inlet
Remove accumulated sediments at the inlet and those trapped in the spillway	Periodic	Annually or as needed	Surface, spillway and inlet
Technical inspection of inlet and outlet structures and spillway for blockages	Inspection	Six-monthly and after heavy rains	Surface
Inspection of the infiltration surface	Inspection	Six-monthly and after heavy rains	Surface
Irrigation to maintain good vegetation density	Periodic	Monthly	Vegetation
Cutting vegetation and removing weeds	Periodic	Monthly	Vegetation
Replant poorly vegetated areas	Corrective	When necessary	Vegetation
Inspection of the basin for puddles showing evidence of clogged areas	Inspection	Annually	Surface
Regular check for sediment accumulation and erosion	Inspection	Annually	Surface, spillway, inlet and vegetation
Verification that no settling is happening	Inspection	Annually	Surface
Inspection to evaluate emptying time	Inspection	Six-monthly and after heavy rains	Surface
Fill in eroded areas and improve erosion protection if necessary	Corrective	When necessary	Surface, spillway, inlet and vegetation
Perform permeability test to determine need for corrective action	Inspection	Biennially	Surface, gravel
Inspect slopes and verify topography	Occasional	Biennially	Surface
Level basin base and reinstate design levels	Occasional	When necessary	Surface
Reconstruction at the end of lifespan	Corrective	At end of lifespan (30 years	Surface, vegetation, inlet, spillway, filter medium, gravel and drainage

ANNEX Nº7: REFERENCES

SUSTAINABLE DRAINAGE SYSTEMS (SUDS) IN INDUSTRIAL AREAS:
APPLICATION TO THE INDUSTRIAL AREA IN QUART DE POBLET (VALENCIA)

Author:
ÁLVAREZ PÉREZ, Ana
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MÁSTER UNIVERSITARIO EN INGENIERÍA DE CAMINOS, CANALES Y PUERTOS
CURSO: 2020-2021
ESCUELA TÉCNICA SUPERIOR DE INGENIERÍA DE CAMINOS, CANALES Y PUERTOS
UNIVERSIDAD POLITÉNICA DE VALENCIA

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DOCUMENT 2. MAPS

SUSTAINABLE DRAINAGE SYSTEMS (SUDS) IN INDUSTRIAL AREAS:
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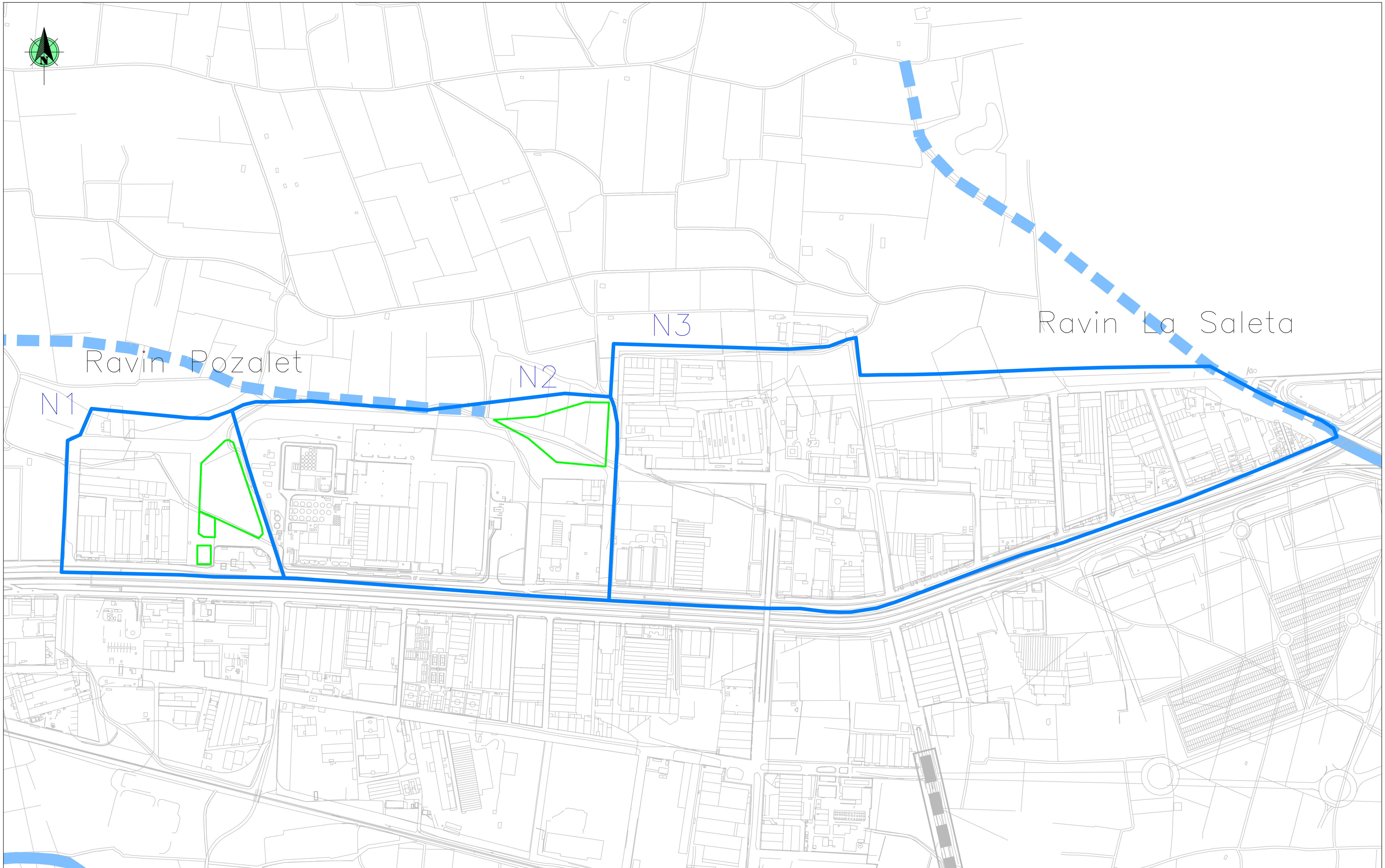
UNIVERSITAT POLITÈCNICA DE VALÈNCIA

Summary

UNIVERSIDAD POLITÉCNICA DE VALENCIA MASTER EN INGENIERÍA DE CAMINOS, CANALES Y PUERTOS	PROJECT AUTHOR: ANA ALVAREZ PEREZ	PROJECT TITLE: Sustainable Drainage Systems (SuDS) in industrial areas: Application to the industrial area in Quart de Poblet (Valencia)	MAP TITLE: LOCATION OF THE STUDY CASE	DATE: 11/02/2021	SCALE: VARIOUS MAP NUMBER: 1



UNIVERSIDAD POLITÉCNICA DE VALENCIA

MASTER EN INGENIERÍA DE CAMINOS, CANALES Y PUERTOS

PROJECT AUTHOR:

ANA ALVAREZ PEREZ

PROJECT TITLE:

Sustainable Drainage Systems (SuDS) in industrial areas:
Application to the industrial area in Quart de Poblet (Valencia)

MAP TITLE:

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DATE:

11/02/2021

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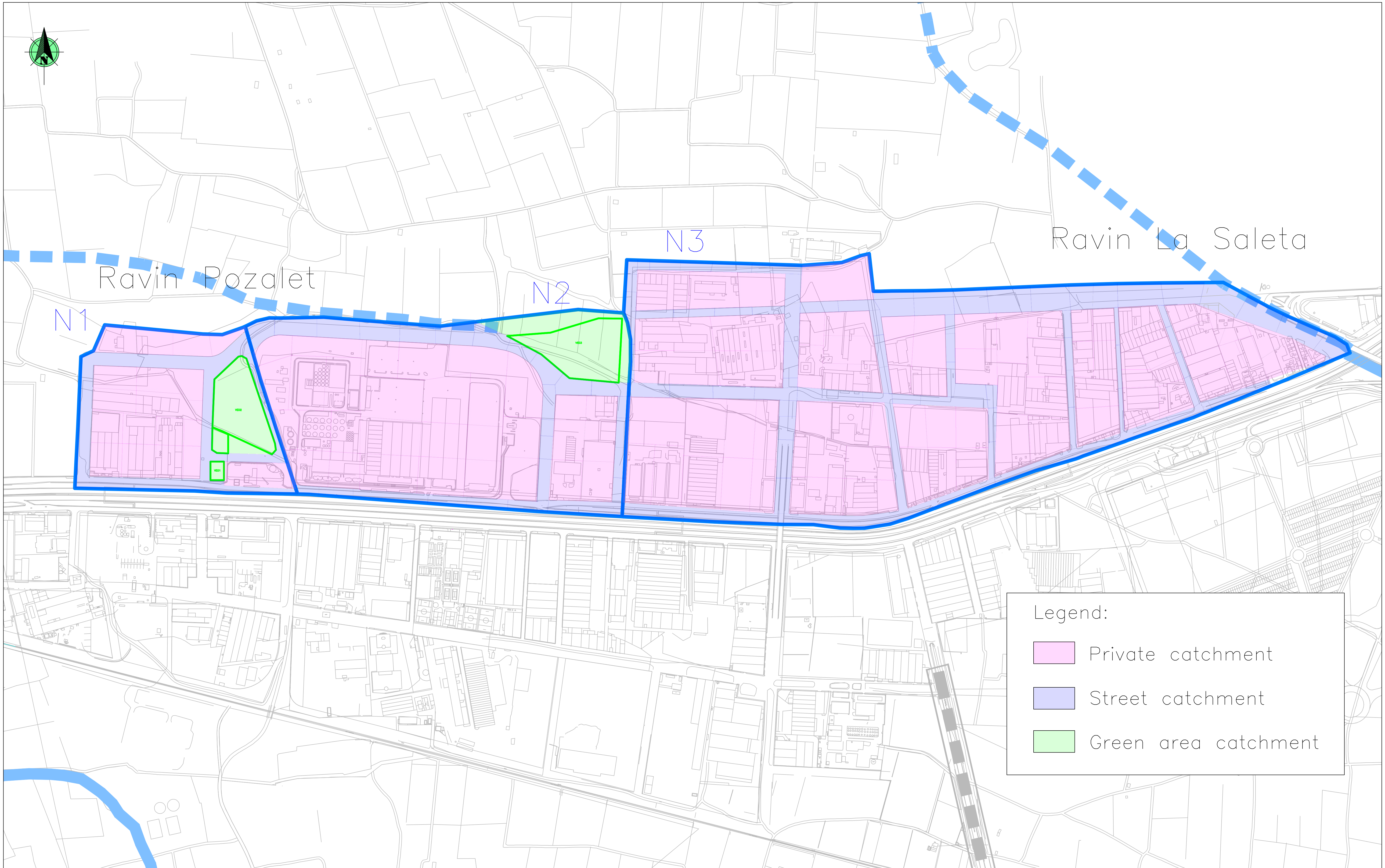

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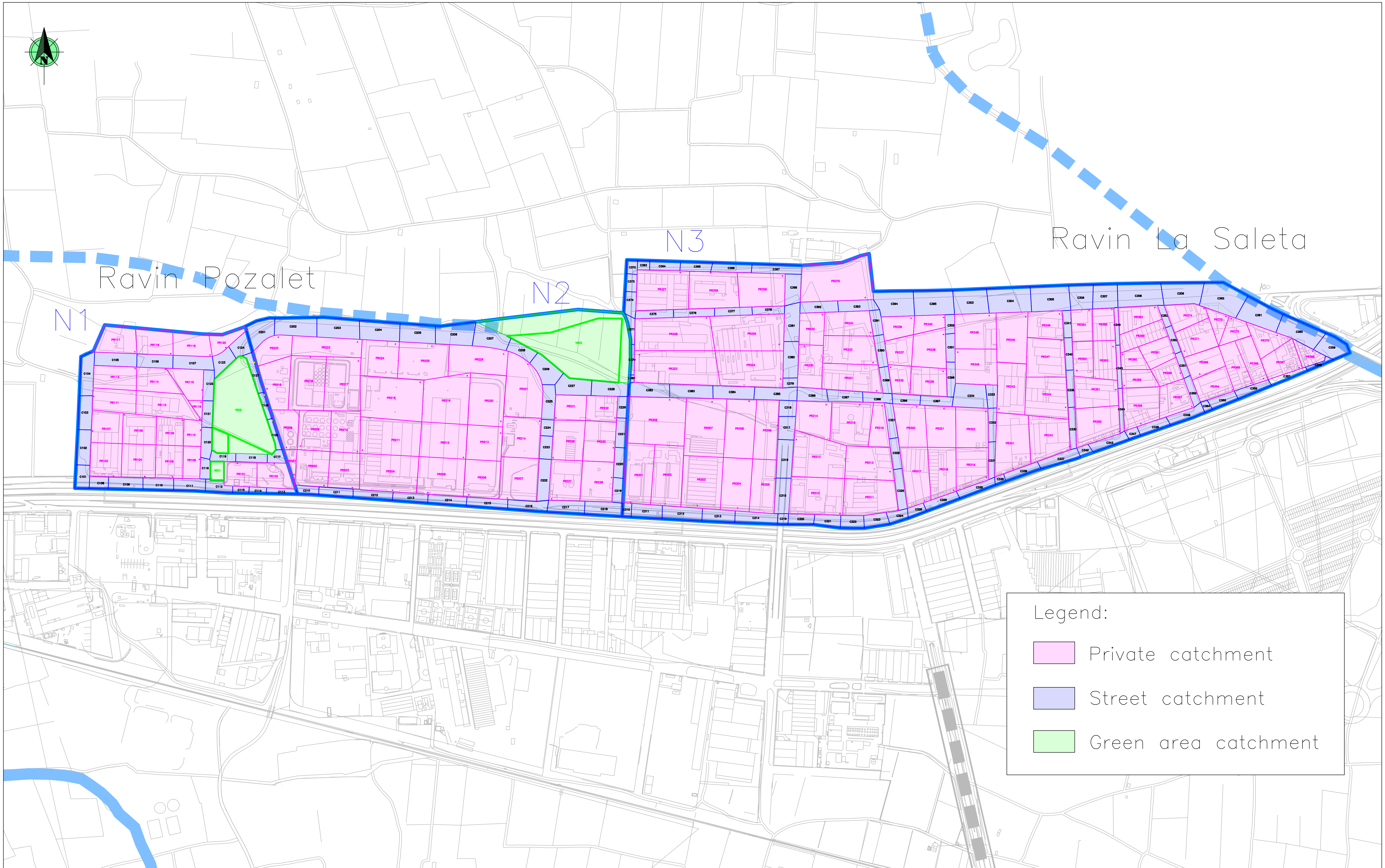
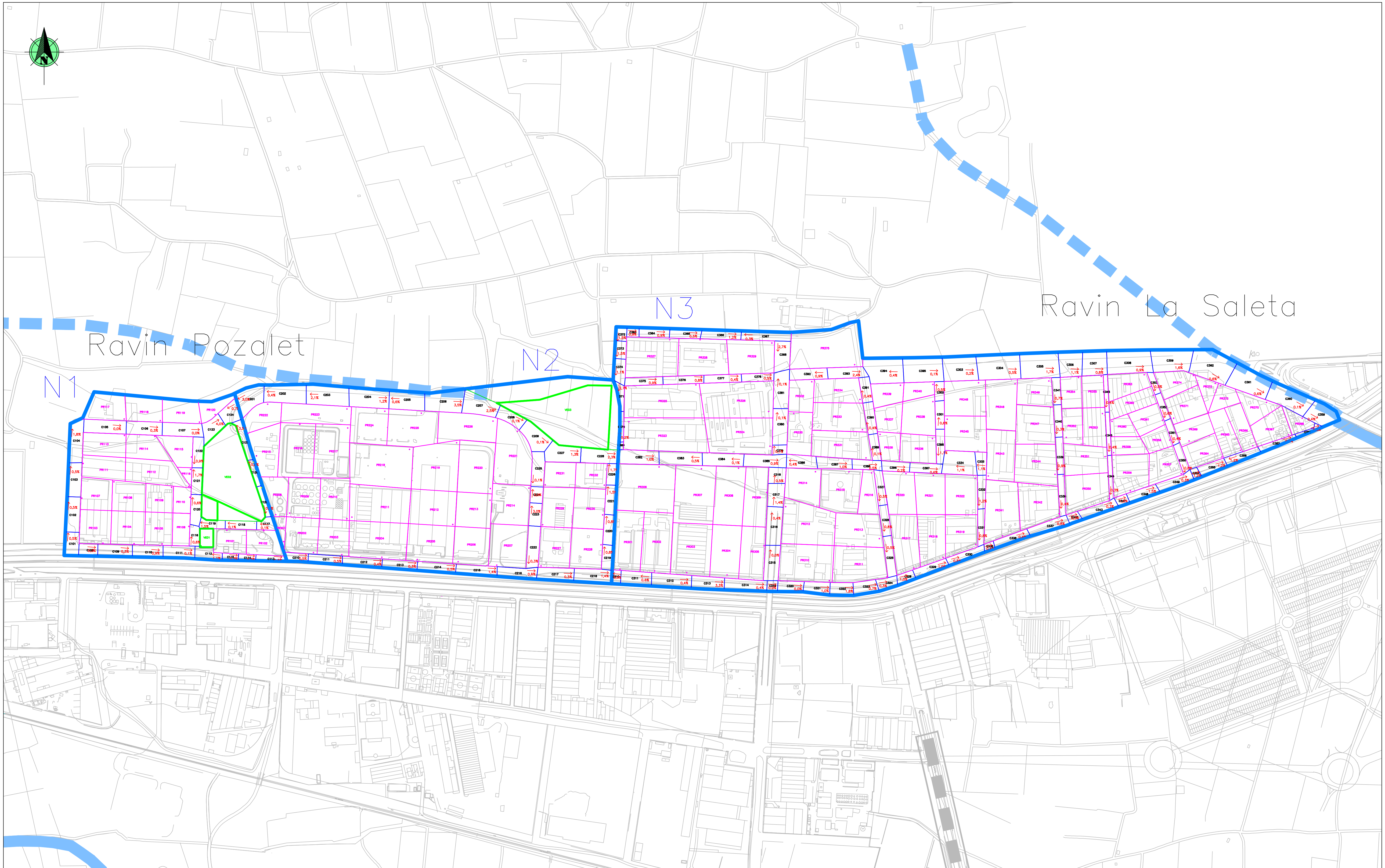


UNIVERSIDAD POLITÉCNICA DE VALENCIA MASTER EN INGENIERÍA DE CAMINOS, CANALES Y PUERTOS	PROJECT AUTHOR: **ANA ALVAREZ PEREZ**	PROJECT TITLE: Sustainable Drainage Systems (SuDS) in industrial areas: Application to the industrial area in Quart de Poblet (Valencia)	MAP TITLE: **SUBCATCHMENTS DIVISION**	DATE: **11/02/2021**	SCALE: **1:8000** MAP NUMBER: **4**



UNIVERSIDAD POLITÉCNICA DE VALENCIA

MASTER EN INGENIERÍA DE CAMINOS, CANALES Y PUERTOS

PROJECT AUTHOR:

ANA ALVAREZ PEREZ

PROJECT TITLE:

Sustainable Drainage Systems (SuDS) in industrial areas:
Application to the industrial area in Quart de Poblet (Valencia)

MAP TITLE:

LONGITUDINAL SLOPES

DATE:

11/02/2021

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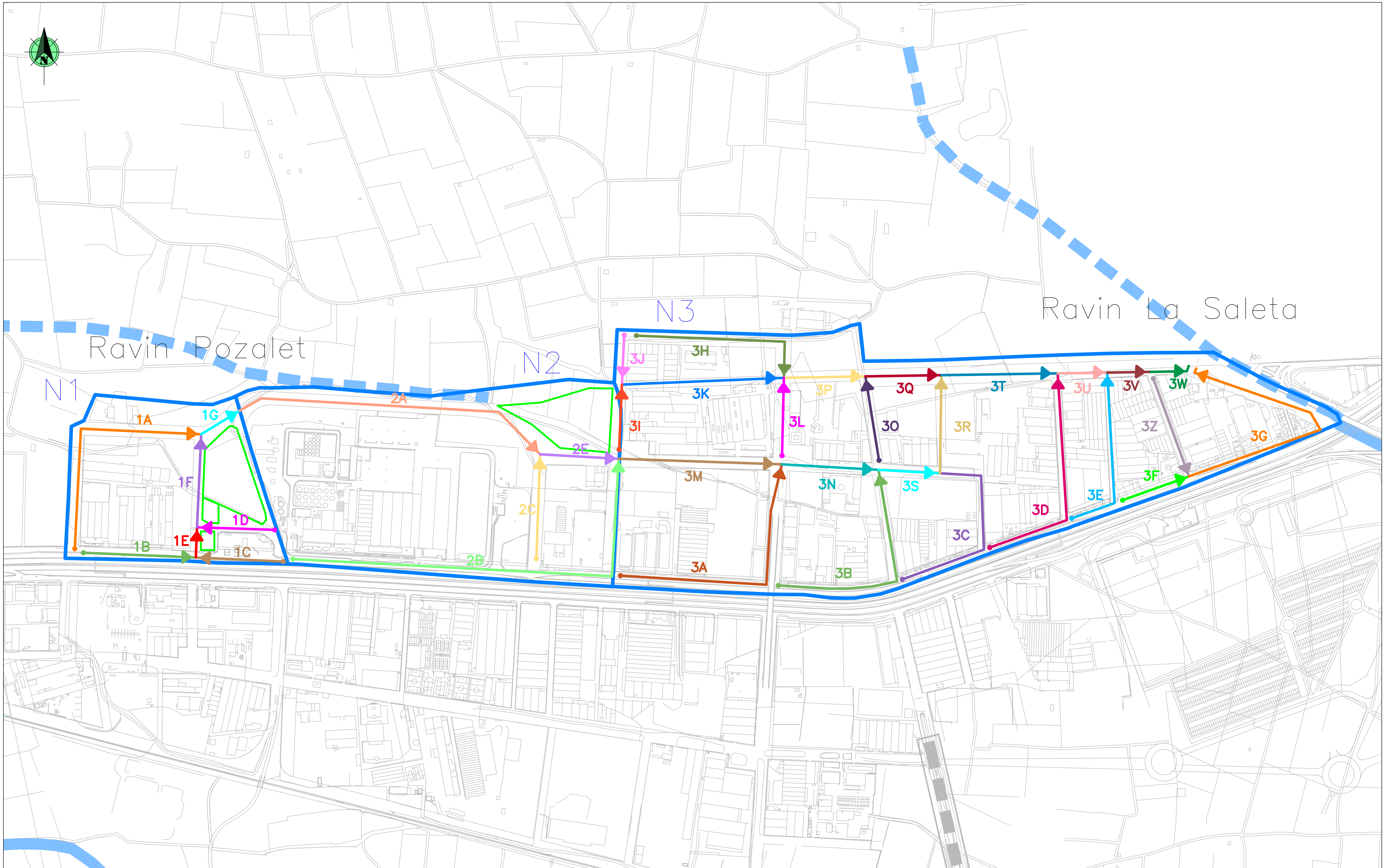
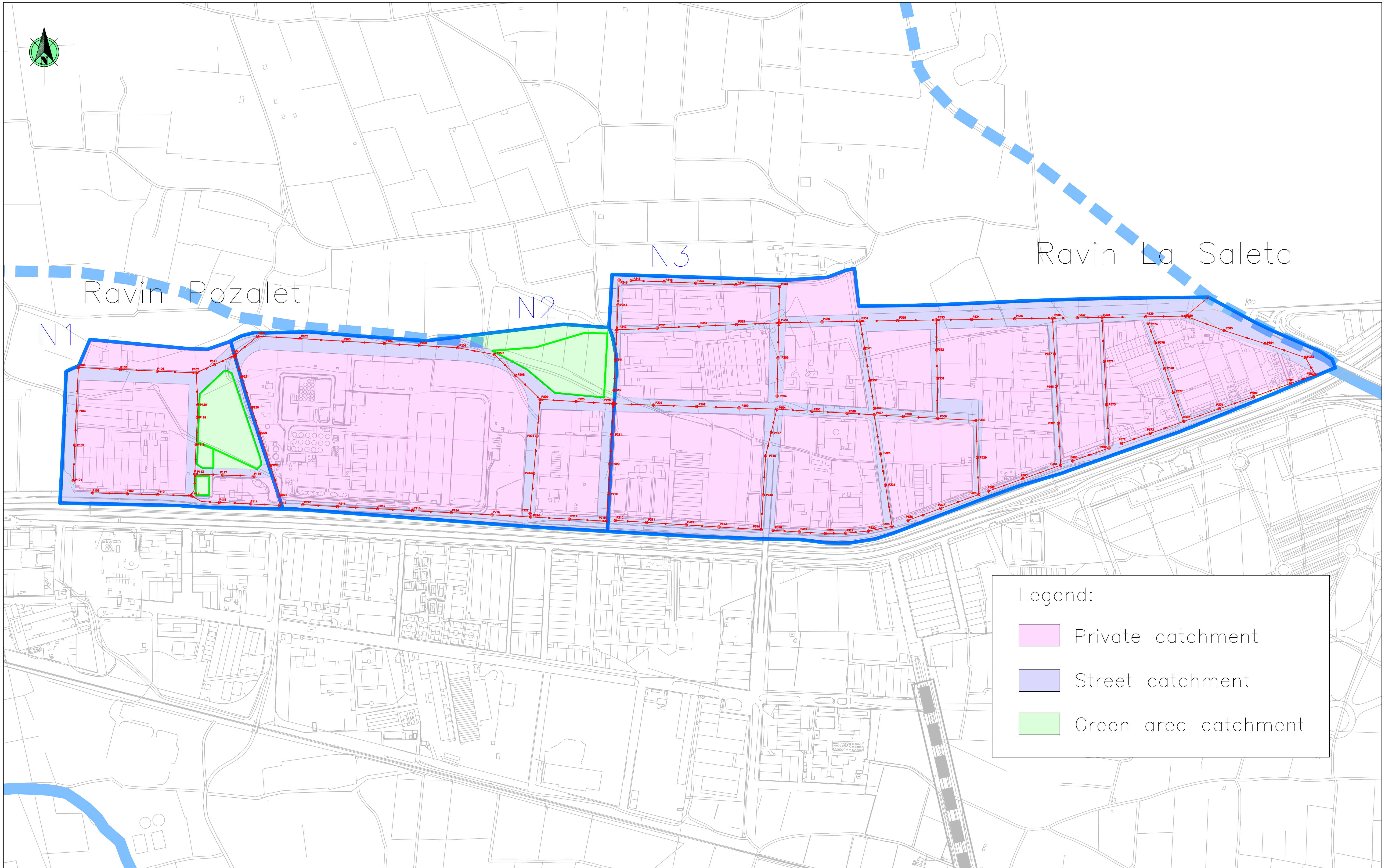
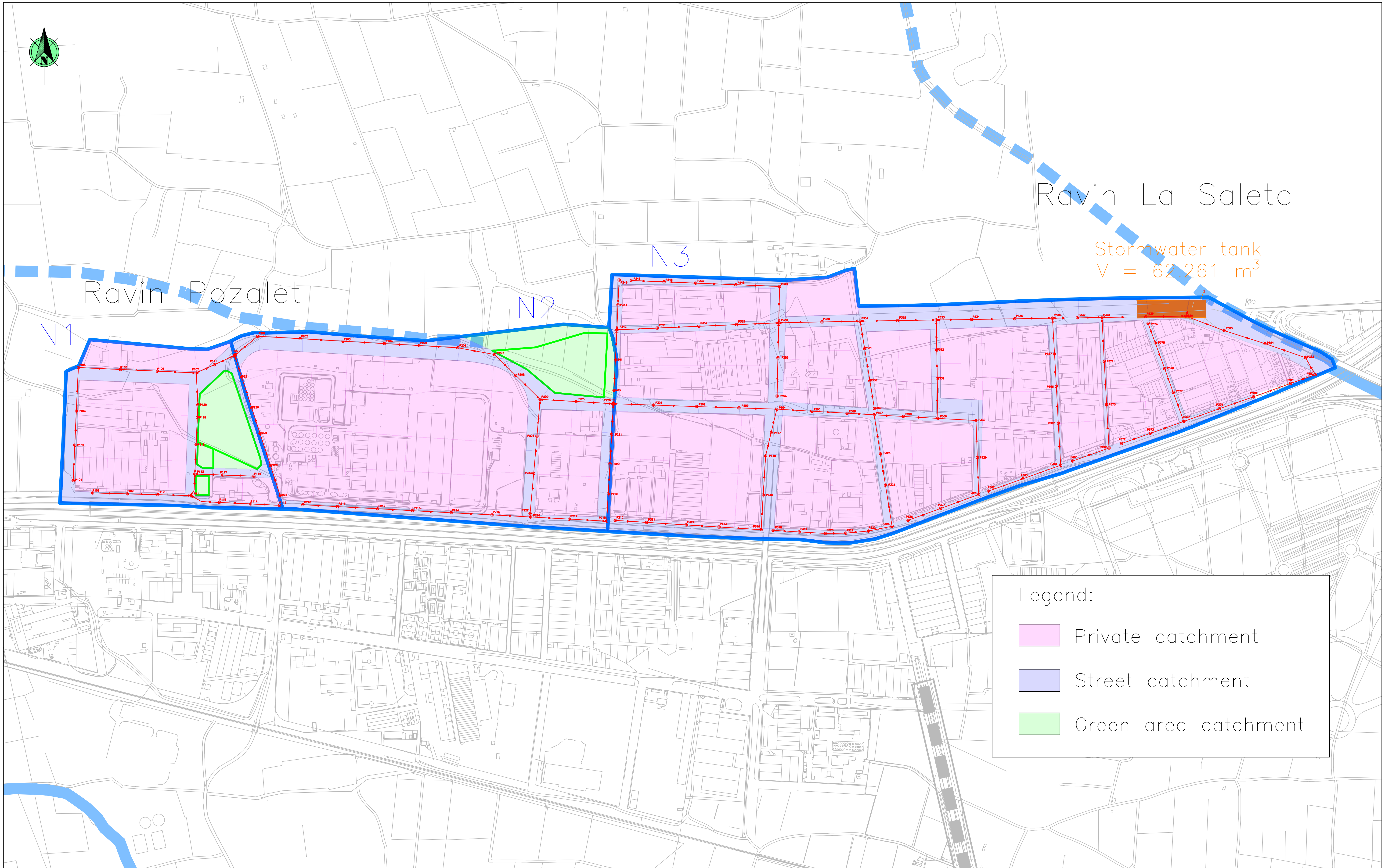


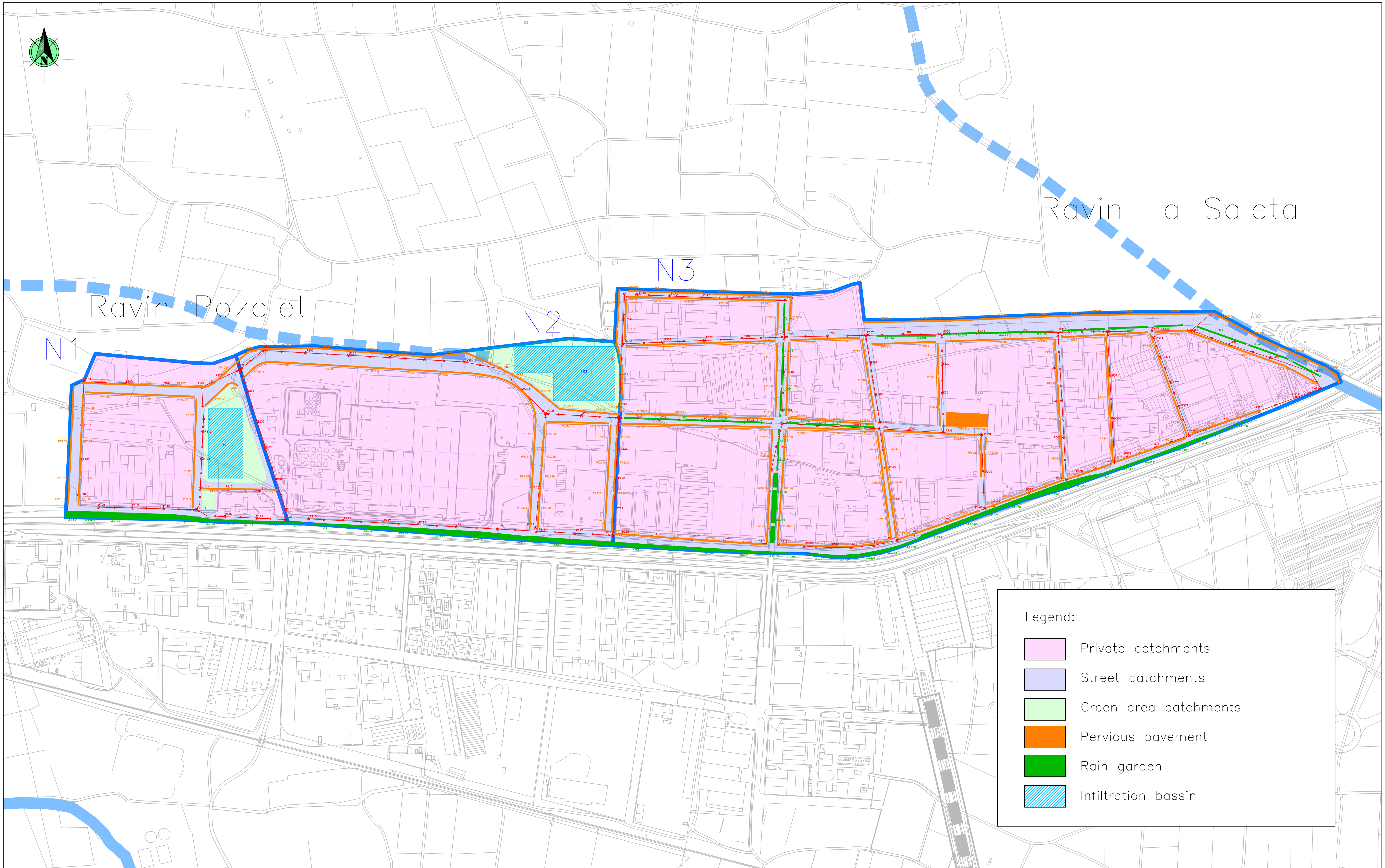
UNIVERSIDAD POLITÉCNICA DE VALENCIA MASTER EN INGENIERÍA DE CAMINOS, CANALES Y PUERTOS	PROJECT AUTHOR: **ANA ALVAREZ PEREZ**	PROJECT TITLE: Sustainable Drainage Systems (SuDS) in industrial areas: Application to the industrial area in Quart de Poblet (Valencia)	MAP TITLE: **DIRECTIONS OF COLLECTORS SEGMENTS**	DATE: **11/02/2021**	SCALE: **1:8000** MAP NUMBER: **6**



UNIVERSIDAD POLITÉCNICA DE VALENCIA MASTER EN INGENIERÍA DE CAMINOS, CANALES Y PUERTOS	PROJECT AUTHOR: **ANA ALVAREZ PEREZ**	PROJECT TITLE: Sustainable Drainage Systems (SuDS) in industrial areas: Application to the industrial area in Quart de Poblet (Valencia)	MAP TITLE: **ALTERNATIVE 1. CONVENTIONAL DRAINAGE SYSTEM**	DATE: **11/02/2021**	SCALE: **1:8000** MAP NUMBER: **7**



UNIVERSIDAD POLITÉCNICA DE VALENCIA MASTER EN INGENIERÍA DE CAMINOS, CANALES Y PUERTOS	PROJECT AUTHOR: **ANA ALVAREZ PEREZ**	PROJECT TITLE: Sustainable Drainage Systems (SuDS) in industrial areas: Application to the industrial area in Quart de Poblet (Valencia)	MAP TITLE: **ALTERNATIVE 2. CONVENTIONAL DRAINAGE SYSTEM WITH A STORMWATER TANK**	DATE: **11/02/2021**	SCALE: **1:8000** MAP NUMBER: **8**



UNIVERSIDAD POLITÉCNICA DE VALENCIA

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PROJECT AUTHOR:

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PROJECT TITLE:

Sustainable Drainage Systems (SuDS) in industrial areas: Application to the industrial area in Quart de Poblet (Valencia)

MAP TITLE:

ALTERNATIVE 3. SUSTAINABLE DRAINAGE SYSTEM

DATE:

11/02/2021

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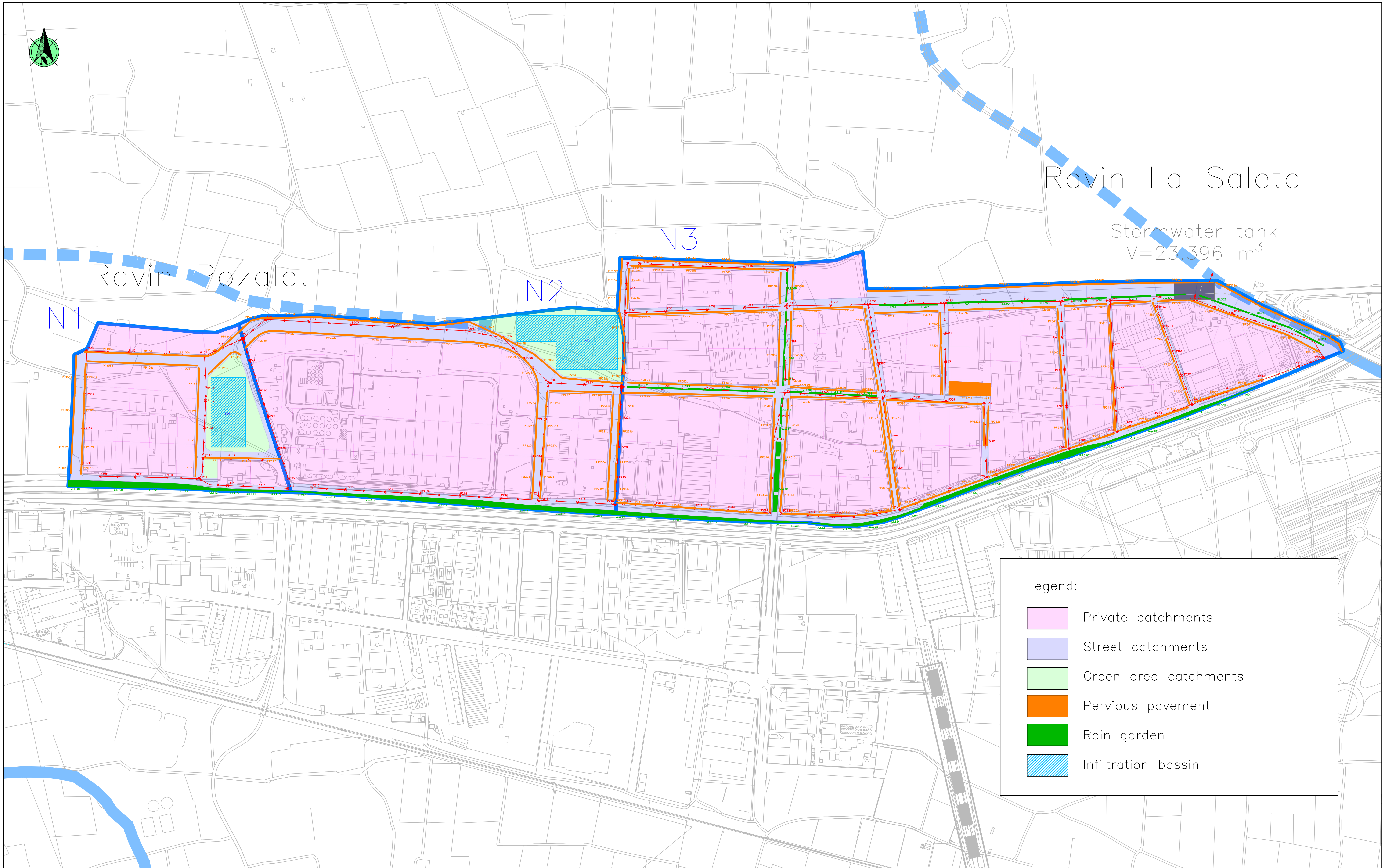

UNIVERSIDAD POLITÉCNICA DE VALENCIA

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PROJECT AUTHOR: **ANA ALVAREZ PEREZ**

PROJECT TITLE: Sustainable Drainage Systems (SuDS) in industrial areas: Application to the industrial area in Quart de Poblet (Valencia)

MAP TITLE: **ALTERNATIVE 4. SUSTAINABLE DRAINAGE SYSTEM WITH STORMWATER TANK**

DATE: **11/02/2021**

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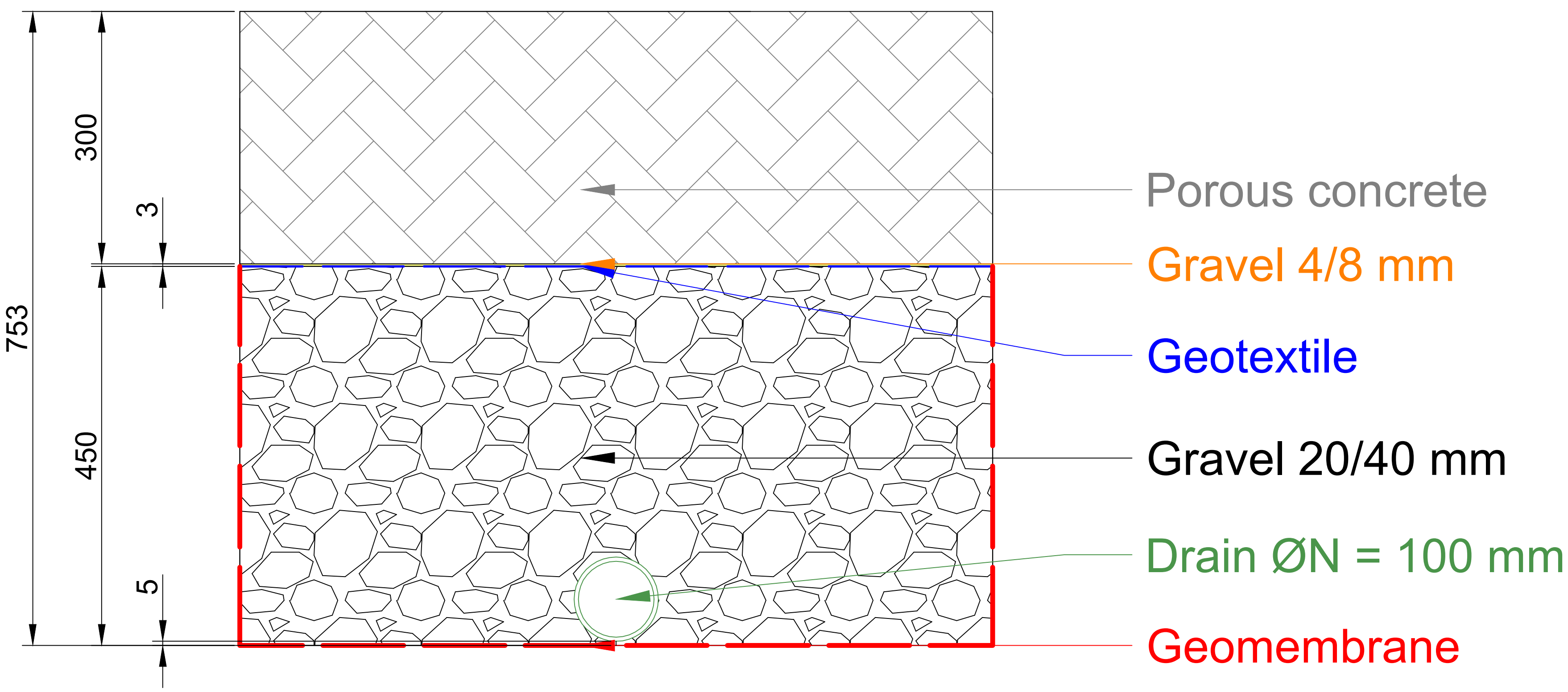
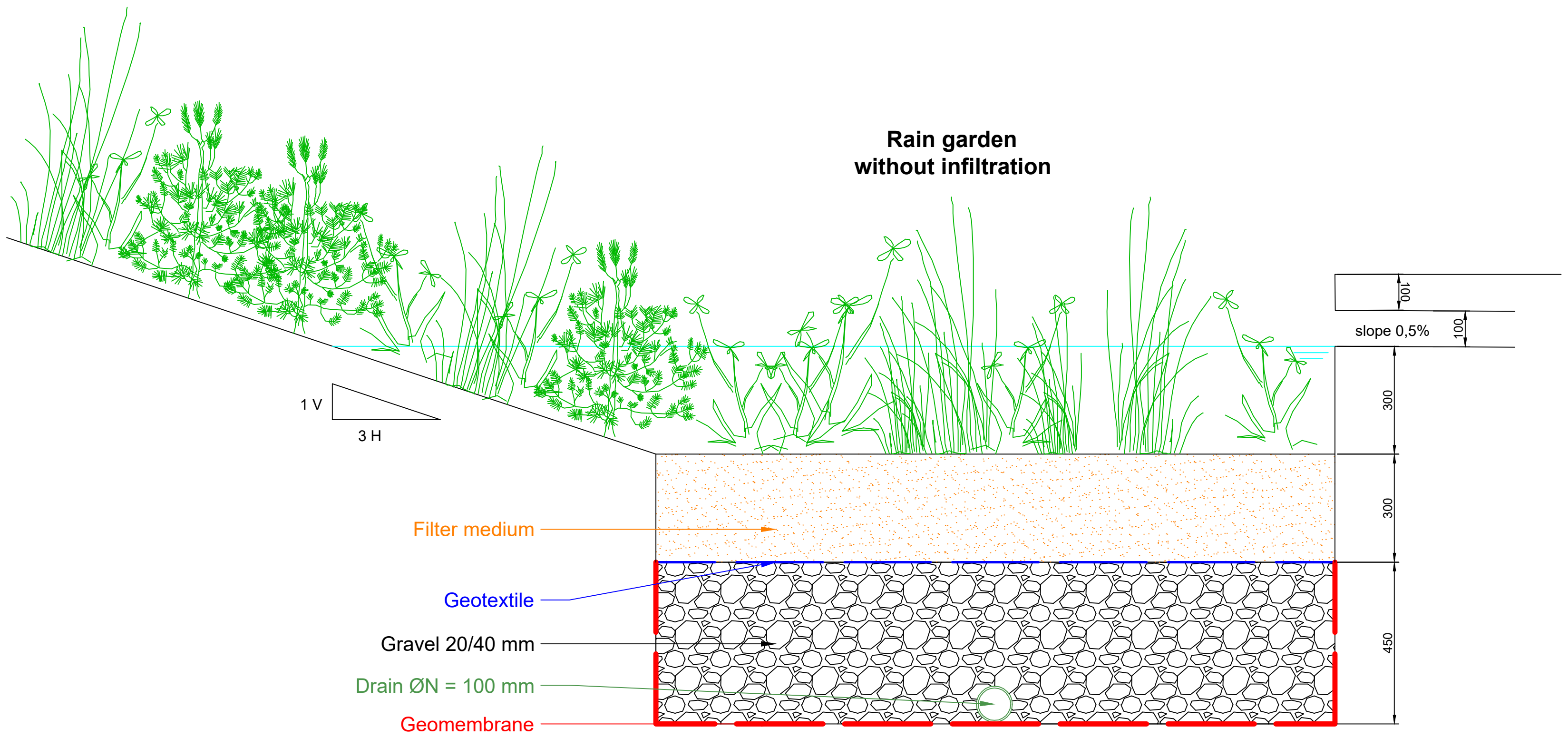
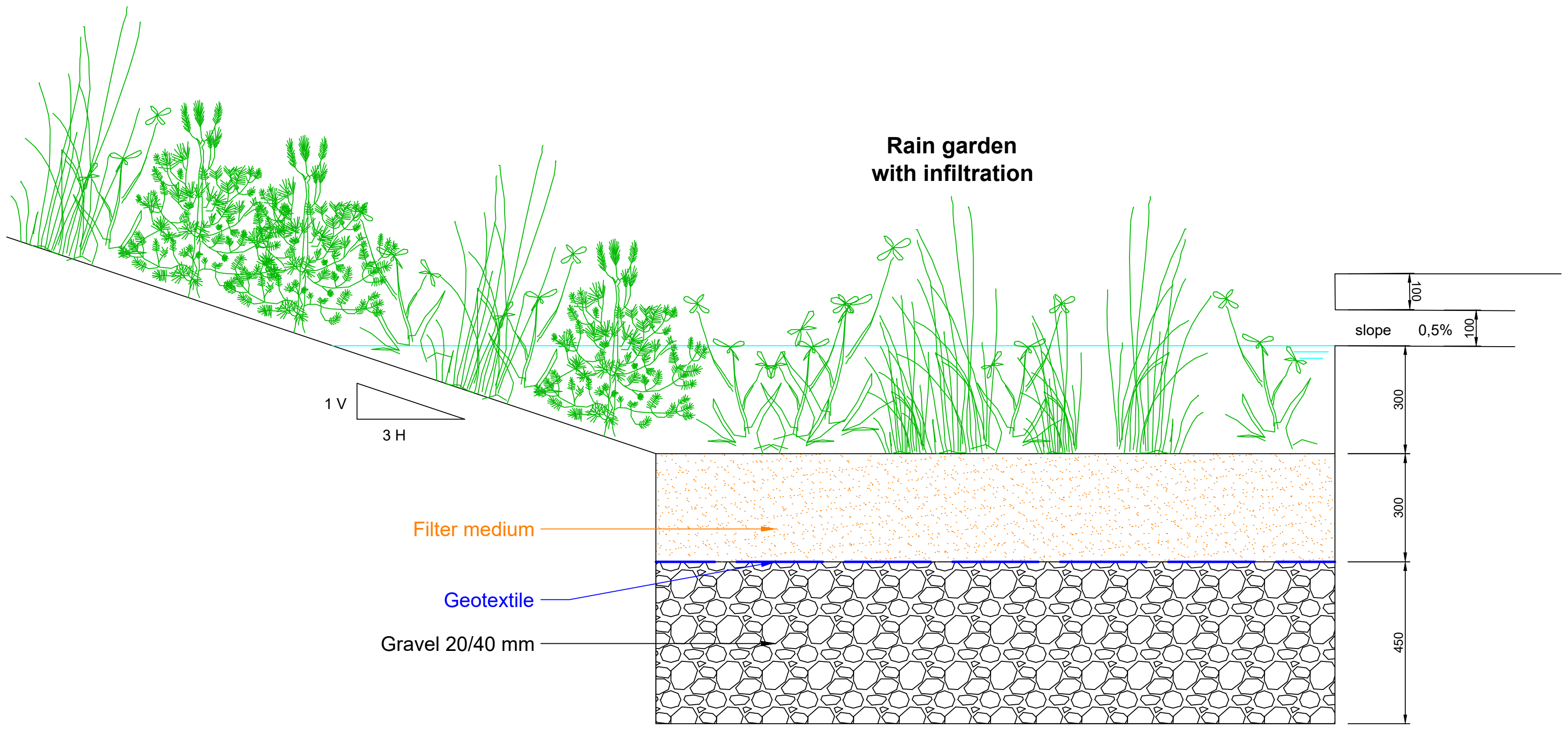
Pervious Pavement



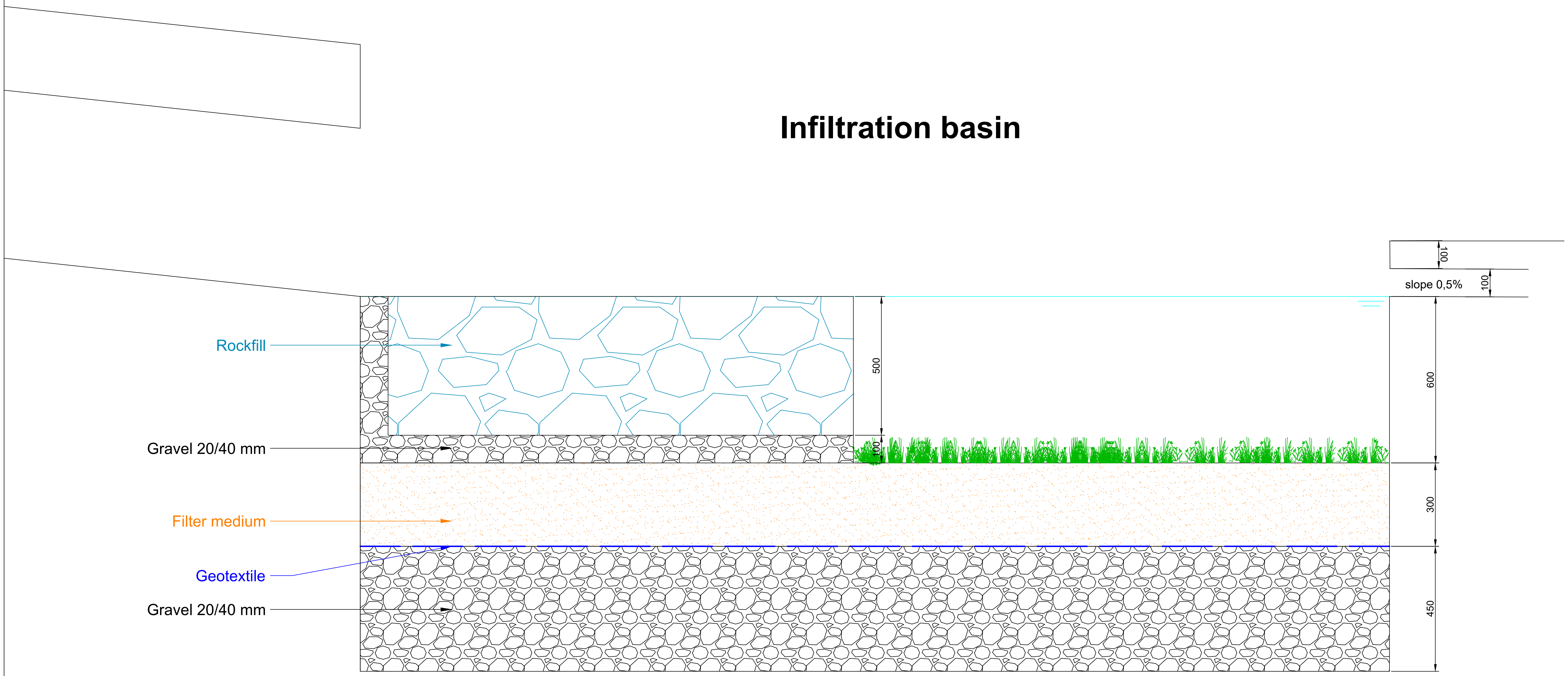
UNIVERSIDAD POLITÉCNICA DE VALENCIA MASTER EN INGENIERÍA DE CAMINOS, CANALES Y PUERTOS	PROJECT AUTHOR: **ANA ALVAREZ PEREZ**	PROJECT TITLE: Sustainable Drainage Systems (SuDS) in industrial areas: Application to the industrial area in Quart de Poblet (Valencia)	MAP TITLE: **CROSS-SECTION OF PERVIOUS PAVEMENT**	DATE: **11/02/2021**	SCALE: **1:8** MAP NUMBER: **11**





UNIVERSIDAD POLITÉCNICA DE VALENCIA MASTER EN INGENIERÍA DE CAMINOS, CANALES Y PUERTOS	PROJECT AUTHOR: **ANA ALVAREZ PEREZ**	PROJECT TITLE: Sustainable Drainage Systems (SuDS) in industrial areas: Application to the industrial area in Quart de Poblet (Valencia)	MAP TITLE: **CROSS-SECTION OF RAIN GARDEN**	DATE: **11/02/2021**	SCALE: **1:20** MAP NUMBER: **12**

Infiltration basin



UNIVERSIDAD POLITÉCNICA DE VALENCIA MASTER EN INGENIERÍA DE CAMINOS, CANALES Y PUERTOS	PROJECT AUTHOR: **ANA ALVAREZ PEREZ**	PROJECT TITLE: Sustainable Drainage Systems (SuDS) in industrial areas: Application to the industrial area in Quart de Poblet (Valencia)	MAP TITLE: **CROSS-SECTION OF INFILTRATION BASIN**	DATE: **11/02/2021**	SCALE: **1:15** MAP NUMBER: **13**	

DOCUMENT 3. ECONOMIC ASSESSMENT

SUSTAINABLE DRAINAGE SYSTEMS (SUDS) IN INDUSTRIAL AREAS:
APPLICATION TO THE INDUSTRIAL AREA IN QUART DE POBLET (VALENCIA)

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Summary

1 OBJECT

The purpose of this document is to define an approximate economic assessment of the total cost of the works to be carried out, proposed as a solution to the problems presented in the industrial estate of Quart de Poblet, Valencia.

To carry out this economic evaluation, it has been defined the work units that make up the execution of the project, as well as the unit prices. Subsequently, the units of measurement are presented and, by means of these two groups of information, we can obtain the partial evaluation of the cost of each action that will make up the action of the project.

The sum total of the partial evaluations will make up the total evaluation of the work, which corresponds to the direct costs. In addition, it is necessary to add the indirect costs which correspond to a percentage of the direct costs. These indirect costs are overhead as an industrial profit with a value of 13% and 6%, respectively. The final result will be obtained from the sum of direct and indirect costs, plus the value added tax which is up to a 21%.

It is important to comment on the estimated nature of the following economic valuation, which allows us to study the economic viability of the proposed solution and thus comply with the scope of this work.

The information corresponding to the unit prices of this document has been obtained from the Institut Valencià de l'Edificació (IVE) which presents its database of 2020 prices for the three municipalities that make up the Comunidad Valenciana, the price base provided by the CYPE software and the database of price units provided by the Madrid Torwnhall.

2 UNIT PRICE AND QUANTITIES

2.1 Earthworks

Code	Unit	Tittle	Description	Unit price	Quantity
CHAPTER 1. EARTHWORKS					
EW01	m3	**Trench excavation**	Excavation of a trench with shoring in a traffic area by mechanical methods, including loading of material and its intermediate stockpiling or transport to a landfill site within a distance of less than 10 km	6,69 €	67.523,09
EW02	m3	**Open-cast excavation**	Open-cast excavation of land for clearing by mechanical means, including the loading of material and its intermediate stockpiling or its transport to a landfill site within a distance of less than 10 km	2,53 €	107.976,38
EW03	m3	**Transport to final destination**	Transport of products resulting from excavation and demolition to final destination, by authorised transporter, considering round trip, with a tipper truck of up to 15 t and auxiliary equipment, measured on profile (not including unloading costs)	5,24 €	50.180,00
EW04	m3	**Landfill fee**	Discharge of excavation and demolition waste to landfill, including the fee and paving	13,39 €	50.180,00
EW05	m3	**Backfilling trench**	Backfilling of trenches with selected earth from the excavation itself, and compaction in successive 25 cm thick layers with mechanical means, until reaching a dry density of no less than 90% of the maximum density obtained in the Modified Proctor test, carried out according to UNE 103501. The price does not include the Modified Proctor test	5,39 €	34.436,49

2.2 Hydraulic system

Code	Unit	Tittle	Description	Unit price	Quantity
CHAPTER 2. HYDRAULIC SYSTEM					
Subchapter 2.1. Pipe Network					
PN01	m	**PVC pipe DN 400**	Collector buried in non-aggressive ground, made of double-walled PVC pipe, corrugated on the outside and smooth on the inside, tile colour RAL 8023, nominal diameter 400 mm, nominal annular stiffness 8 kN/m². The price includes the equipment necessary to move and position the elements on site, but does not include excavation and main backfill	83,45 €	600,30
PN02	m	**PVC pipe DN 500**	Collector buried in non-aggressive ground, made of double-walled PVC pipe, corrugated on the outside and smooth on the inside, tile colour RAL 8023, nominal diameter 500 mm, nominal annular stiffness 8 kN/m². The price includes the equipment necessary to move and position the elements on site, but does not include excavation and main backfill	135,89 €	396,50
PN03	m	**PVC pipe DN 630**	Collector buried in non-aggressive ground, made of double-walled PVC pipe, corrugated on the outside and smooth on the inside, tile colour RAL 8023, nominal diameter 630 mm, nominal annular stiffness 8 kN/m². The price includes the equipment necessary to move and position the elements on site, but does not include excavation and main backfill	155,55 €	896,20
PN04	m	**PVC pipe DN 800**	Collector buried in non-aggressive ground, made of double-walled PVC pipe, corrugated on the outside and smooth on the inside, tile colour RAL 8023, nominal diameter 800 mm, nominal annular stiffness 8 kN/m². The price includes the equipment necessary to move and position the elements on site, but does not include excavation and main backfill	240,91 €	1.761,50
PN05	m	**PVC pipe DN 1000**	Collector buried in non-aggressive ground, made of double-walled PVC pipe, corrugated on the outside and smooth on the inside, tile colour RAL 8023, nominal diameter 1000 mm, nominal annular stiffness 8 kN/m². The price includes the equipment necessary to move and position the elements on site, but does not include excavation and main backfill	346,93 €	1.930,70
PN06	m	**PVC pipe DN 1200**	Collector buried in non-aggressive ground, made of double-walled PVC pipe, corrugated on the outside and smooth on the inside, tile colour RAL 8023, nominal diameter 1200 mm, nominal annular stiffness 8 kN/m². The price includes the equipment necessary to move and position the elements on site, but does not include excavation and main backfill	472,12 €	1.048,50
PN07	m	**PVC pipe DN 1300**	Collector buried in non-aggressive ground, made of double-walled rigid PVC pipes, helically shaped with a profile that presents a wall structured by means of "T" shaped stiffeners and reinforcement with galvanized steel profile, nominal diameter 1300 mm, nominal annular stiffness 8 kN/m². The price includes the equipment necessary to move and position the elements on site, but does not include excavation and main backfill	566,54 €	1.104,00
PN08	m	**PVC pipe DN 1400**	Collector buried in non-aggressive ground, made of double-walled rigid PVC pipes, helically shaped with a profile that presents a wall structured by means of "T" shaped stiffeners and reinforcement with galvanized steel profile, nominal diameter 1400 mm, nominal annular stiffness 8 kN/m². The price includes the equipment necessary to move and position the elements on site, but does not include excavation and main backfill	651,53 €	92,90
PN09	m	**PVC pipe DN 1500**	Collector buried in non-aggressive ground, made of double-walled rigid PVC pipes, helically shaped with a profile that presents a wall structured by means of "T" shaped stiffeners and reinforcement with galvanized steel profile, nominal diameter 1500 mm, nominal annular stiffness 8 kN/m². The price includes the equipment necessary to move and position the elements on site, but does not include excavation and main backfill	749,25 €	279,70
PN10	m	**PVC pipe DN 1700**	Collector buried in non-aggressive ground, made of double-walled rigid PVC pipes, helically shaped with a profile that presents a wall structured by means of "T" shaped stiffeners and reinforcement with galvanized steel profile, nominal diameter 1700 mm, nominal annular stiffness 8 kN/m². The price includes the equipment necessary to move and position the elements on site, but does not include excavation and main backfill	861,64 €	172,30
PN11	m	**PVC pipe DN 1800**	Collector buried in non-aggressive ground, made of double-walled rigid PVC pipes, helically shaped with a profile that presents a wall structured by means of "T" shaped stiffeners and reinforcement with galvanized steel profile, nominal diameter 1800 mm, nominal annular stiffness 8 kN/m². The price includes the equipment necessary to move and position the elements on site, but does not include excavation and main backfill	990,89 €	387,80
PN12	m	**PVC pipe DN 1900**	Collector buried in non-aggressive ground, made of double-walled rigid PVC pipes, helically shaped with a profile that presents a wall structured by means of "T" shaped stiffeners and reinforcement with galvanized steel profile, nominal diameter 1900 mm, nominal annular stiffness 8 kN/m². The price includes the equipment necessary to move and position the elements on site, but does not include excavation and main backfill	1.139,52 €	193,70

PN13	m	**PVC pipe DN 2000**	Collector buried in non-aggressive ground, made of double-walled rigid PVC pipes, helically shaped with a profile that presents a wall structured by means of "T" shaped stiffeners and reinforcement with galvanized steel profile, nominal diameter 2000 mm, nominal annular stiffness 8 kN/m². The price includes the equipment necessary to move and position the elements on site, but does not include excavation and main backfill	1.310,45 €	338,80
PN14	m	**PVC pipe DN 2200**	Collector buried in non-aggressive ground, made of double-walled rigid PVC pipes, helically shaped with a profile that presents a wall structured by means of "T" shaped stiffeners and reinforcement with galvanized steel profile, nominal diameter 2200 mm, nominal annular stiffness 8 kN/m². The price includes the equipment necessary to move and position the elements on site, but does not include excavation and main backfill	1.507,02 €	188,60
PN15	m	**PVC pipe DN 2300**	Collector buried in non-aggressive ground, made of double-walled rigid PVC pipes, helically shaped with a profile that presents a wall structured by means of "T" shaped stiffeners and reinforcement with galvanized steel profile, nominal diameter 2300 mm, nominal annular stiffness 8 kN/m². The price includes the equipment necessary to move and position the elements on site, but does not include excavation and main backfill	1.733,07 €	181,30
PN16	m	**PVC pipe DN 2400**	Collector buried in non-aggressive ground, made of double-walled rigid PVC pipes, helically shaped with a profile that presents a wall structured by means of "T" shaped stiffeners and reinforcement with galvanized steel profile, nominal diameter 2400 mm, nominal annular stiffness 8 kN/m². The price includes the equipment necessary to move and position the elements on site, but does not include excavation and main backfill	1.993,03 €	220,70
PN17	m	**PVC pipe DN 2500**	Collector buried in non-aggressive ground, made of double-walled rigid PVC pipes, helically shaped with a profile that presents a wall structured by means of "T" shaped stiffeners and reinforcement with galvanized steel profile, nominal diameter 2500 mm, nominal annular stiffness 8 kN/m². The price includes the equipment necessary to move and position the elements on site, but does not include excavation and main backfill	2.291,99 €	144,40
PN18	m	**PVC pipe DN 2600**	Collector buried in non-aggressive ground, made of double-walled rigid PVC pipes, helically shaped with a profile that presents a wall structured by means of "T" shaped stiffeners and reinforcement with galvanized steel profile, nominal diameter 2600 mm, nominal annular stiffness 8 kN/m². The price includes the equipment necessary to move and position the elements on site, but does not include excavation and main backfill	2.635,78 €	222,80
PN19	m	**PVC pipe DN 2900**	Collector buried in non-aggressive ground, made of double-walled rigid PVC pipes, helically shaped with a profile that presents a wall structured by means of "T" shaped stiffeners and reinforcement with galvanized steel profile, nominal diameter 2700 mm, nominal annular stiffness 8 kN/m². The price includes the equipment necessary to move and position the elements on site, but does not include excavation and main backfill	3.031,15 €	263,00
PN20	m	**PVC pipe DN 3000**	Collector buried in non-aggressive ground, made of double-walled rigid PVC pipes, helically shaped with a profile that presents a wall structured by means of "T" shaped stiffeners and reinforcement with galvanized steel profile, nominal diameter 3000 mm, nominal annular stiffness 8 kN/m². The price includes the equipment necessary to move and position the elements on site, but does not include excavation and main backfill	3.485,82 €	301,50
Subchapter 2.2. Registration system					
RS01	Ud	**Manholes**	Manhole, 1.00 m inner diameter and 2.1 m inner useful height, made of prefabricated mass concrete elements, on a 25 cm thick reinforced concrete slab HA-30/B/20/IIb+Qb lightly reinforced with electrowelded mesh, with a locking circular cover and cast iron frame class D-400 according to UNE-EN 124, installed in roadways, including pedestrian areas, or parking areas for all types of vehicles. The price includes the equipment necessary to move and position the elements on site, but does not include excavation or backfilling	625,98 €	106,00
RS02	Ud	**Registration well**	Brick-built registration well, with internal dimensions 125x125x200 cm, with prefabricated reinforced concrete cover, on a mass concrete slab. The price does not include excavation or backfilling.	616,55 €	32,00
Subchapter 2.3. Reinforced concrete					
RC01	m3	**Concrete HA-30/P/20/IIa+Qb**	Concrete HA-30/P/20/IIa+Qb produced in the factory with SR cement, and poured from a truck	116,21 €	15.743,51
RC02	kg	**Steel B500S**	UNE-EN 10080 B 500 S steel for reinforcement work (cutting, bending and shaping of elements) in site workshop and assembly. Including tying wire and spacers.	1,85 €	67.509,16

2.3 Sustainable drainage system

Code	Unit	Tittle	Description	Unit price	Quantity
CHAPTER 3. SUSTAINABLE DRAINAGE SYSTEMS					
Subchapter 3.1. Pervious pavement					
PP01	m2	**Pervious concrete pavement**	Continuous porous concrete pavement HM-D-330/F/8 "LAFARGEHOLCIM", with low fines content, manufactured in central, grey colour, with a flexural strength of 2 N/mm², a compressive strength of 45 N/mm² and a drainage capacity of 150 mm/h, with 25% voids and slip resistance Rd>45 according to UNE 41901 EX, slipperiness class 3 according to CTE, 300 mm thick, laid on a layer of granular material. The price does not include the granular material layer	54,54 €	44.739,56
PP02	m3	**Gravel 4/8**	Supply, spreading and compaction of 4 to 8 mm gravel, placed in trenches or surfaces for drainage, in 20 cm layers, measured on profile	10,96 €	134,22
PP03	m2	**Non-woven geotextile 25,2 KN/m**	Non-woven geotextile composed of needle-punched polypropylene fibres, with a longitudinal tear strength of 25.2 kN/m and a transverse tear strength of 25.6 kN/m, laid on the ground.	2,12 €	44.739,56
PP04	m3	**Gravel 20/40**	Backfilling of trenches with 20 to 40 mm diameter gravel. Including tape or marking indicating the installation.	17,94 €	20.132,80
PP05	m2	**Impervious geomembrane**	Homogeneous impervious geomembrane of plasticised PVC, weather-resistant, 1.2 mm thick, grey in colour, with a density of 1240 kg/m³ according to UNE-EN ISO 1183, CBR punching resistance of 1.8 kN according to UNE-EN ISO 12236 and a tear strength of more than 40 kN/m, laid with overlaps, without adhering to the substrate, on a non-woven geotextile composed of needle-punched polypropylene fibres, with a longitudinal tear strength of 27.9 kN/m, a transverse tear strength of 31.6 kN/m	15,73 €	57.294,20
PP06	m	**PVC drain DN 100**	Buried collector of horizontal sewerage network, with manholes, for the evacuation of wastewater and/or rainwater, formed by smooth PVC pipe, SN-4 series, nominal annular stiffness 4 kN/m², 100 mm external diameter, glued by means of adhesive. Including cleaning liquid and adhesive for PVC pipes and fittings. The price does not include excavation and main backfill.	15,69 €	13.949,60
Subchapter 3.2. Rain Garden					
RG01	m3	**Filter medium**	Backfill of filter medium, with a mixture of sand, topsoil and organic matter (content between 3-5%, with void ratio > 30 % and hydraulic conductivity of 100-300 mm/h, including paving and compacting, according to the conditions of the Technical Specifications.	20,81 €	8.074,43
RG02	m2	**Non-woven geotextile 8 KN/m**	Non-woven geotextile composed of needle-punched polypropylene fibres, with a longitudinal tear strength of 8.0 kN/m and a transverse tear strength of 9.0 kN/m, laid on the ground.	0,75 €	26.914,75
RG03	m3	**Gravel 20/40**	Backfilling of trenches with 20 to 40 mm diameter gravel. Including tape or marking indicating the installation.	17,94 €	12.111,64
RG04	m2	**Impervious geomembrane**	Homogeneous impervious geomembrane of plasticised PVC, weather-resistant, 1.2 mm thick, grey in colour, with a density of 1240 kg/m³ according to UNE-EN ISO 1183, CBR punching resistance of 1.8 kN according to UNE-EN ISO 12236 and a tear strength of more than 40 kN/m, laid with overlaps, without adhering to the substrate, on a non-woven geotextile composed of needle-punched polypropylene fibres, with a longitudinal tear strength of 27.9 kN/m, a transverse tear strength of 31.6 kN/m	15,73 €	10.962,92
RG05	m	**PVC drain DN 100**	Buried collector of horizontal sewerage network, with manholes, for the evacuation of wastewater and/or rainwater, formed by smooth PVC pipe, SN-4 series, nominal annular stiffness 4 kN/m², 100 mm external diameter, glued by means of adhesive. Including cleaning liquid and adhesive for PVC pipes and fittings. The price does not include excavation and main backfill.	15,69 €	1.209,35
RG06	m2	**Vegetation**	Supply and planting of an perennial bush, Abelia floribunda type or similar, with a density of 6 units/m2, including remodelling, digging and fertilising of the soil, distribution of the plant, mulching and first watering.	6,16 €	26.914,75
Subchapter 3.3. Infiltration basin					
IN01	m3	**Filter medium**	Backfill of filter medium, with a mixture of sand, topsoil and organic matter (content between 3-5%, with void ratio > 30 % and hydraulic conductivity of 100-300 mm/h, including paving and compacting, according to the conditions of the Technical Specifications.	20,81 €	10.621,62
IN02	m3	**Rockfill**	Backfill of limestone block rockfill, faced, placed with backhoe excavator on chains with rockfill pincer	84,64 €	15,00
IN03	m3	**Gravel 20/40**	Backfilling of trenches with 20 to 40 mm diameter gravel. Including tape or marking indicating the installation.	17,94 €	15.935,42
IN04	m2	**Sowing**	Sowing of a rustic grass seed mixture with high resistance to trampling, including soil preparation with a motorized plough, distribution of complex fertiliser, profiling and rolling. Sowing of the mixture indicates mulching and first watering.	2,32 €	35.405,39

3 ECONOMIC ASSESSMENT

The related total costs for each work unit are represented in this section.

Code	Tittle	Cost
CHAPTER 1. EARTHWORKS		
EW01	Trench excavation	451.729,47 €
EW02	Open-cast excavation	273.180,24 €
EW03	Transport to final destination	262.943,19 €
EW04	Landfill fee	671.910,16 €
EW05	Backfilling trench	185.612,69 €
CHAPTER 2. HYDRAULIC SYSTEM		
Subchapter 2.1. Pipe Network		
PN01	PVC pipe DN 400	50.095,04 €
PN02	PVC pipe DN 500	53.880,39 €
PN03	PVC pipe DN 630	139.403,91 €
PN04	PVC pipe DN 800	424.362,97 €
PN05	PVC pipe DN 1000	669.817,75 €
PN06	PVC pipe DN 1200	495.017,82 €
PN07	PVC pipe DN 1300	625.464,58 €
PN08	PVC pipe DN 1400	60.526,73 €
PN09	PVC pipe DN 1500	209.566,47 €
PN10	PVC pipe DN 1700	148.461,02 €
PN11	PVC pipe DN 1800	384.266,75 €
PN12	PVC pipe DN 1900	220.725,48 €
PN13	PVC pipe DN 2000	443.980,70 €
PN14	PVC pipe DN 2200	284.223,65 €
PN15	PVC pipe DN 2300	314.205,78 €
PN16	PVC pipe DN 2400	439.862,10 €
PN17	PVC pipe DN 2500	330.962,85 €
PN18	PVC pipe DN 2600	587.252,77 €
PN19	PVC pipe DN 2900	797.193,00 €
PN20	PVC pipe DN 3000	1.050.976,21 €
Subchapter 2.2. Registration system		
RS01	Manholes	66.353,88 €
RS02	Registration wells	19.729,60 €
Subchapter 2.3. Reinforced concrete		
RC01	Concrete HA-30/P/20/IIa+Qb	1.829.552,74 €
RC02	Steel B500S	124.891,94 €
CHAPTER 3. SUSTAINABLE DRAINAGE SYSTEMS		
Subchapter 3.1. Pervious pavement		
PP01	Pervious concrete pavement	2.440.095,60 €
PP02	Gravel 4/8	1.471,04 €
PP03	Non-woven geotextile 25,2 KN/m	94.847,87 €
PP04	Gravel 20/40	361.182,47 €
PP05	Impervious geomembrane	901.237,77 €
PP06	PVC drain DN 100	218.869,22 €
Subchapter 3.2. Rain garden		
RG01	Filter medium	168.028,78 €
RG02	Non-woven geotextile 8 KN/m	20.186,06 €
RG03	Gravel 20/40	217.282,78 €
RG04	Impervious geomembrane	172.446,65 €
RG05	PVC drain DN 100	18.974,70 €
RG06	Vegetation	165.794,86 €
Subcatchment 3.3 Infiltration basin		
IN01	Filter medium	221.035,83 €
IN02	Rockfill	1.269,60 €
IN03	Gravel 20/40	285.881,51 €
IN04	Sowing	82.140,50 €

4 SUMMARY ECONOMIC ASSESSMENT

Finally, a summary of the economic assessment by chapter is presented, distinguishing between direct and indirect costs. The budget of the project amounts to TWENTY-FOUR MILLION FOUR HUNDRED AND FIFTY-NINE THOUSAND FOUR HUNDRED AND THIRTY EUROS AND TWENTY SEVEN CENTS.

Description	Economic assessment
Chapter 1. Earthworks	**1.845.375,75 €**
Chapter 2. Hydraulic system	**9.770.774,11 €**
Subchapter 2.1. Pipe network	7.730.245,94 €
Subchapter 2.2. Registration system	86.083,48 €
Subchapter 2.3. Reinforced concrete elements	1.954.444,69 €
Chapter 3. Sustainable drainage systems	**5.370.745,25 €**
Subchapter 3.1. Pervious pavement	4.017.703,96 €
Subchapter 3.2. Rain garden	762.713,84 €
Subchapter 3.3. Infiltration basin	590.327,45 €
TOTAL DIRECT COSTS	**16.986.895,11 €**
Overhead costs (13%)	2.208.296,36 €
Industrial benefit (6%)	1.019.213,71 €
TOTAL DIRECT AND INDIRECT COSTS	**20.214.405,18 €**
Value Added Tax (21%)	4.245.025,09 €
TOTAL BUDGET	**24.459.430,27 €**

Anexo al Trabajo Fin de Grado/Máster

Relación del TFG/TFM "Sustainable Drainage Systems (SuDS) in industrial areas: Application to the industrial area in Quart de Poblet (Valencia)" con los Objetivos de Desarrollo Sostenible de la Agenda 2030.

Grado de relación del trabajo con los Objetivos de Desarrollo Sostenible (ODS).

Objetivos de Desarrollo Sostenibles	**Alto**	**Medio**	**Bajo**	**No Procede**
ODS 1. **Fin de la pobreza.**				**X**
ODS 2. **Hambre cero.**				**X**
ODS 3. **Salud y bienestar.**				**X**
ODS 4. **Educación de calidad.**				**X**
ODS 5. **Igualdad de género.**				
ODS 6. **Agua limpia y saneamiento.**	**X**			
ODS 7. **Energía asequible y no contaminante.**				
ODS 8. **Trabajo decente y crecimiento económico.**				
ODS 9. **Industria, innovación e infraestructuras.**		**X**		
ODS 10. **Reducción de las desigualdades**.				
ODS 11. **Ciudades y comunidades sostenibles**.	**X**			
ODS 12. **Producción y consumo responsables.**				**X**
ODS 13. **Acción por el clima.**	**X**			
ODS 14. **Vida submarina.**				**X**
ODS 15. **Vida de ecosistemas terrestres.**	**X**			
ODS 16. **Paz, justicia e instituciones sólidas.**				**X**
ODS 17. **Alianzas para lograr objetivos.**				**X**

Descripción de la alineación del TFG/M con los ODS con un grado de relación más alto.

El presente trabajo final de máster, en un contexto en el que se busca el compromiso de comunidad científica en la mejora de la sociedad en el marco de los ODS, ha perseguido el cumplimiento de ciertos objetivos mencionados anteriormente. En particular, los objetivos perseguidos en este trabajo han sido:

- Objetivo 6. Garantizar la disponibilidad de agua y su gestión sostenible y el saneamiento para todos.
- Objetivo 9. Construir infraestructuras resilientes, promover la industrialización sostenible y fomentar la innovación.
- Objetivo 11. Lograr que las ciudades sean más inclusivas, seguras, resilientes y sostenibles.
- Objetivo 13. Adoptar medidas urgentes para combatir el cambio climático y sus efectos.
- Objetivo 15. Gestionar sosteniblemente los bosques, luchar contra la desertificación, detener e invertir la degradación de las tierras, detener la pérdida de biodiversidad.

A continuación, se exponen las contribuciones del trabajo para la consecución de cada uno de los objetivos, mencionando las metas específicas que se han buscado alcanzar:

Objetivo 6. Garantizar la disponibilidad de agua y su gestión sostenible y el saneamiento para todos.

Uno de los principales objetivos a los que contribuye el trabajo es a la gestión sostenible del agua mediante sistemas de drenaje sostenibles. En concreto, las metas indicadas por este objetivo que se buscan son la 6.3, 6.5 y 6.b.

6.3 De aquí a 2030, mejorar la calidad del agua reduciendo la contaminación, eliminando el vertimiento y minimizando la emisión de productos químicos y materiales peligrosos, reduciendo a la mitad el porcentaje de aguas residuales sin tratar y aumentando considerablemente el reciclado y la reutilización sin riesgos a nivel mundial

6.5 De aquí a 2030, implementar la gestión integrada de los recursos hídricos a todos los niveles, incluso mediante la cooperación transfronteriza, según proceda

6.b Apoyar y fortalecer la participación de las comunidades locales en la mejora de la gestión del agua y el saneamiento

9 INDUSTRIA, INNOVACIÓN E INFRAESTRUCTURAS

Objetivo 9. Construir infraestructuras resilientes, promover la industrialización sostenible y fomentar la innovación.

Con una menor relación que el objetivo anterior, se busca en este objetivo el desarrollo e impulso de infraestructuras sostenibles como son los sistemas de drenaje sostenibles. Además, de cierta forma se menciona las áreas industriales que, aunque no se entre en detalle de desarrollo de actividades industriales sostenibles, se trata el desarrollo de medidas sostenibles en el ámbito de la industrialización. La meta a la que hace mención este trabajo es la 9.1.

9.1 Desarrollar infraestructuras fiables, sostenibles, resilientes y de calidad, incluidas infraestructuras regionales y transfronterizas, para apoyar el desarrollo económico y el bienestar humano, haciendo especial hincapié en el acceso asequible y equitativo para todos

11 CIUDADES Y COMUNIDADES SOSTENIBLES

Objetivo 11. Lograr que las ciudades sean más inclusivas, seguras, resilientes y sostenibles.

Este objetivo se persigue desde diferentes frentes, el primero es el que tiene relación con las inundaciones y las consecuencias ocasionadas por los desastres naturales, aunque también se trata el uso eficiente de recursos y la implementación de políticas relacionada con ellos. Las metas de este objetivo perseguidas en el trabajo son la 11.5 y 11.b.

11.5 De aquí a 2030, reducir significativamente el número de muertes causadas por los desastres, incluidos los relacionados con el agua, y de personas afectadas por ellos, y reducir considerablemente las pérdidas económicas directas provocadas por los desastres en comparación con el producto interno bruto mundial, haciendo hincapié en la protección de los pobres y las personas en situaciones de vulnerabilidad

11.b De aquí a 2020, aumentar considerablemente el número de ciudades y asentamientos humanos que adoptan e implementan políticas y planes integrados para promover la inclusión, el uso eficiente de los recursos, la mitigación del cambio climático y la adaptación a él y la resiliencia ante los desastres, y desarrollar y poner en práctica, en consonancia con el Marco de Sendai para la Reducción del Riesgo de Desastres 2015-2030, la gestión integral de los riesgos de desastre a todos los niveles

Objetivo 13. Adoptar medidas urgentes para combatir el cambio climático y sus efectos.

El cambio climático es uno de los mayores detonantes de las acciones preventivas y correctivas en ingeniería hidráulica, sobre todo con los eventos extraordinarios que cada vez son más frecuentes. Por ello se persigue este objetivo, ya no solo para mitigar los efectos que tienen las consecuencias del cambio climático, como son los eventos extremos, sino también para actuar para reducirlo. Las metas a las que se hace mención en este trabajo son la 13.1, 13.2 y 13.3.

13.1 Fortalecer la resiliencia y la capacidad de adaptación a los riesgos relacionados con el clima y los desastres naturales en todos los países

13.2 Incorporar medidas relativas al cambio climático en las políticas, estrategias y planes nacionales

13.3 Mejorar la educación, la sensibilización y la capacidad humana e institucional respecto de la mitigación del cambio climático, la adaptación a él, la reducción de sus efectos y la alerta temprana

Objetivo 15. Gestionar sosteniblemente los bosques, luchar contra la desertificación, detener e invertir la degradación de las tierras, detener la pérdida de biodiversidad.

Trabajando en los beneficios de los sistemas de drenaje sostenibles, se ha hecho mención a su carácter favorable en cuanto a los ecosistemas y la biodiversidad. Siendo este objetivo uno de los que en mayor forma se consiguen cumplir. En el presente trabajo se han perseguido las metas 15.3 y 15.a

15.3 Para 2030, luchar contra la desertificación, rehabilitar las tierras y los suelos degradados, incluidas las tierras afectadas por la desertificación, la sequía y las inundaciones, y procurar lograr un mundo con una degradación neutra del suelo.

15.a Movilizar y aumentar de manera significativa los recursos financieros procedentes de todas las fuentes para conservar y utilizar de forma sostenible la diversidad biológica y los ecosistemas.