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COORDINATOR Prof. COLTORTI MASSIMO

Evaluation of bedload yield in two small sand-bed rivers of the Ravenna province (Italy)

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Candidate
Dott. Cilli Silvia

Supervisor
Prof. Ciavola Paolo

Co-Supervisors
Prof. Billi Paolo
Prof. Schippa Leonardo

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<<A volte il guerriero della luce si comporta come l'acqua e fluisce tra gli ostacoli che incontra. In certi momenti, resistere significa essere distrutto. Allora egli si adatta alle circostanze. Accetta, senza lagnarsi, che le pietre del cammino traccino la sua rotta attraverso le montagne. In questo consiste la forza dell'acqua: non potrà mai essere spezzata da un martello, o ferita da un coltello. La più potente spada del mondo non potrà mai lasciare alcuna cicatrice sulla sua superficie. L'acqua di un fiume si adatta al cammino possibile, senza dimenticare il proprio obiettivo: il mare. Fragile alla sorgente, a poco a poco acquista la forza dagli altri fiumi che incontra. E, a partire da un certo momento, il suo potere è totale.>>

Paulo Coelho – Manuale del Guerriero della Luce

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Summary

Sediment transport, and bedload transport in particular, is among one of the most dynamic and complex result of hydroclimatic, hydraulic, and geomorphologic processes which occur on a river basin. Sediment production capacity is strictly dependent on the supply of sediment from the catchment which is highly controlled by primary factors such as: climate, basin area, geology and topography. Additionally influenced by human intervention which includes land-use, land cover, presence of infrastructures, gravel mining together with water abstraction for irrigation purposes, sediment transport influences the geomorphic responses of rivers affecting the equilibrium of the fluvial environment from big to small scale. Together with these aspects, in the next future it is expected that even climate change issues further exacerbate the situation. Several are the sampling devices and techniques used in the field to quantify bedload transport. Difficulties liked to their efficiency, together with the request of large human and financial resources, have proven that bedload transport is one of the most difficult fluvial processes to measure. These correlated difficulties have pushed scientists to develop empirical equations for bedload transport prediction. Several formulas on bedload transport have been proposed in the past fifty years based on both laboratory and field studies, though field measurements are very limited. Despite continued efforts made to achieve increasingly accurate and advanced models for bedload transport estimations, still big limitations of bedload transport equations exist. Understanding and quantifying sediment yield is thus becoming increasingly the center of attention of a variety of scientific and societal problems. Between the fields mainly involved there are: geomorphology, civil-and environmental engineering, sedimentary geology and river ecology. One of the most implicated field is coastal geomorphology, especially intended in terms of coastal stability and maintenance. In fact rivers are the main source of sediment to coasts contributing to their formation, stability, preservation and management. Hence the protection and the maintenance of the coastal environment of the Mediterranean Sea which in the recent years has undergone alterations caused by a reduction in sediment delivery from rivers to coastal areas. One of the most effected nations is Italy whose coasts have been subjected to alarming coastal erosion processes, with particular attention devoted to the Emilia-Romagna region. Characterized by tourism and recreational activities which are very common in summer, this sector of the Adriatic coast started to suffer of beach retreat fifty years ago and has worsened in the last decades. Though the causes of such beach retreat may be manifold, preliminary studies showed that a reduction in river sediment supply and the consequent alteration of the beach sediment budget are among the most important. Some soft mitigation measures as sand replenishment and submerged breakwaters have been used but their efficiency is still questionable. Despite a few sporadic field studies on sediment transport in the region, bed load yield data are extremely sporadic and, unfortunately, referred only to a restricted number of rivers. Given the poor knowledge of river supplies at regional scale, this research aims at quantifying the bedload yield to the Romagna beaches through field investigations and measurements on two representative rivers to enlarge the actual dataset. The field data are then used to define a methodological criterion applicable at a regional scale, able to quantify the sediment flux also of the other rivers of the Emilia-Romagna region. By means of hydraulic and hydrological approach and, most of all, by means of bed load transport direct measurement campaigns in the field, this research aims at defining the quantity of sediment supply in the Fiumi Uniti and Savio rivers, equally paradigmatic in terms of mouth morphodynamics. They both flow from the northern Apennines to the Padan plain, outflowing into the Adriatic Sea at Ravenna

province. Focusing particularly on bedload transport, the investigation takes place at the last 10 kilometers of the rivers, i.e. in the vicinity of their mouths. Field campaign have been performed with direct measurement approach (Helley-Smith bedload sampler) which started in 2005-06 by previous authors and completed in 2019, reaching a complete dataset which amounts to 24 and 14 floods on the Fiumi Uniti and Savio, respectively. Repeated samplings carried out during different flood conditions have permitted to calculate bedload rating curves (and annual bedload yields). Additional bathymetric surveys of the river downstream reaches were carried out, revealing the presence of bedform which have been also investigated since they play an important role in the interaction of sediment transport and flow processes. Important components affecting the bedload transport rates and its calculation using the classical bedload equations were also investigated. They include the threshold conditions of incipient bedload motion and the roughness component of dune bedforms. The field data indicate that bedload sediment yield to the Romagna beaches is highly variable and that the Fiumi Uniti bedload transport is higher than the one of the Savio river. Finally, a comparison of the field data with the results of well-known criteria to predict bedload transport rate has been performed to define the best equation to be used at regional scale to quantify bedload supply to beaches in a mid-term scenario.

Riassunto

Il trasporto di sedimenti, e in particolare il trasporto solido (al fondo), è uno dei più dinamici e complessi risultati dei processi idro-climatici, idraulici e geomorfologici che si verificano all'interno di un bacino fluviale. La capacità di produzione sedimentaria dipende strettamente dalla fornitura di sedimenti dal bacino che è altamente controllato da fattori primari quali: clima, area del bacino, geologia e topografia. Ulteriormente influenzato da impatti antropici che includono l'uso del suolo, la copertura del suolo, la presenza di infrastrutture, l'estrazione di ghiaia insieme all'estrazione di acqua a fini di irrigazione, il trasporto di sedimenti influenza le risposte geomorfiche dei fiumi che, a loro volta, influenzano l'equilibrio dell'ambiente fluviale sia a grande che a piccola scala. Insieme a questi aspetti, nel prossimo futuro si prevede anche che le questioni relative ai cambiamenti climatici possano aggravare ulteriormente la situazione. Diversi sono gli strumenti e le tecniche di campionamento utilizzati sul campo per quantificare il trasporto solido. Le difficoltà connesse alla loro efficienza, assieme alla richiesta di grandi risorse umane e finanziarie, hanno dimostrato che il trasporto solido è uno dei processi fluviali più difficili da misurare. Tali difficoltà hanno spinto infatti gli scienziati a sviluppare modelli empirici per la stima del trasporto solido al fondo. Negli ultimi cinquant'anni sono state proposte diverse formule basate fondamentalmente su studi di laboratorio e di campo, pur considerando che le misure dirette di campo siano molto limitate. Nonostante i continui sforzi compiuti per ottenere modelli sempre più precisi e all'avanguardia per le stime del trasporto solido, esistono ancora grandi limiti delle equazioni. Comprendere e quantificare il tasso dei sedimenti trasportati sta diventando sempre più importante per una varietà di problemi sia da un punto di vista scientifico che sociale. Tra i settori maggiormente coinvolti ci sono: la geomorfologia, l'ingegneria civile e quella ambientale, la geologia sedimentaria ed, infine, l'ecologia fluviale. Uno dei campi con maggiori implicazioni è la geomorfologia costiera, specialmente intesa in termini di stabilità e manutenzione costiera. Infatti i fiumi sono la principale fonte di sedimenti per le coste che contribuiscono alla loro formazione, stabilità, conservazione e gestione. A tale proposito è bene considerare la difesa e la conservazione dell'ambiente costiero del Mar Mediterraneo che negli ultimi anni ha subito alterazioni causate da una riduzione di sedimenti fluviali verso le aree costiere. Una delle nazioni più colpite è l'Italia le cui coste sono state sottoposte ad allarmanti processi di erosione costiera, con particolare attenzione alla regione Emilia-Romagna. Caratterizzata da attività turistiche e ricreative molto diffuse in estate, questo settore della costa adriatica ha iniziato a soffrire di fenomeni erosivi circa cinquanta anni fa ed è peggiorato negli ultimi decenni. Sebbene le cause di questo fenomeno erosivo possano essere molteplici, studi preliminari hanno dimostrato che una riduzione dell'apporto sedimentario fluviale e la conseguente alterazione del bilancio sedimentario costiero sono tra le più importanti. A tale proposito sono state adottate alcune misure cautelari come rinascimenti di sabbia e costruzione di barriere frangiflutti sommerse, ciò nonostante la loro efficienza resta ancora discutibile. Sebbene vi siano alcuni sporadici studi sperimentali sugli apporti solidi fluviali nella regione, i dati a disposizione sono estremamente esigui e, sfortunatamente, si riferiscono solo a un numero limitato di fiumi. Data la scarsa conoscenza degli apporti fluviali a scala regionale, questa ricerca mira a quantificare gli apporti solidi dei corsi d'acqua alle spiagge romagnole attraverso indagini sul campo e misurazioni dirette svolte su due fiumi rappresentativi con lo scopo di ampliare il dataset attuale a disposizione. I dati ottenuti dalle indagini in questione sono stati utilizzati per definire un criterio metodologico applicabile a scala regionale, in grado di quantificare l'apporto di sedimenti utilizzabile anche per gli altri fiumi dell'Emilia-Romagna. Mediante un approccio di tipo idraulico ed

idrologico e, soprattutto, mediante campagne di misura diretta del trasporto solido, questa ricerca mira a definire la quantità di sedimenti trasportati al fondo nei Fiumi Uniti e Savio, fiumi che sono considerati ugualmente peculiari in termini di morfodinamica di foce. Entrambi i fiumi scorrono dall'Appennino settentrionale alla Pianura Padana, sfociando nel mare Adriatico in prossimità della provincia di Ravenna. Concentrandosi in particolare sul trasporto solido fluviale, l'indagine si svolge nei pressi degli ultimi 10 chilometri dei corsi d'acqua, vale a dire a ridosso delle zone fociali. Le campagne sono state condotte con un approccio di misurazione diretta (ossia tramite l'utilizzo del campionatore di trasporto solido Helley-Smith) iniziato nel 2005-06 da autori precedenti e completato nel 2019, ottenendo un dataset complessivo che ammonta a 24 e 14 piene campionate su Fiumi Uniti e Savio, rispettivamente. Campionamenti ripetuti effettuati in diverse condizioni di piena hanno permesso di calcolare le scale di deflusso degli apporti solidi (e il carico solido al fondo fluviale annuale). Ulteriori indagini batimetriche sono state condotte in prossimità delle sezioni di misura, rivelando la presenza di forme di fondo che sono state studiate poiché considerati elementi che svolgono un importante ruolo nell'interazione dei processi di trasporto e movimentazione dei sedimenti. Ulteriori approfondimenti sono stati svolti relativamente ai fattori che influenzano il trasporto solido, del quale è stata anche effettuata una stima analizzando alcune classiche equazioni presenti in letteratura. Sono state infatti considerate sia le condizioni di soglia del movimento dei sedimenti sia la componente di rugosità dovuta alla presenza delle forme di fondo, ed in particolare dovuta alle dune. I dati ottenuti indicano che l'apporto solido fluviale dei fiumi studiati è molto variabile ed in particolare il trasporto solido dei Fiumi Uniti è superiore a quello del fiume Savio. Infine, è stato eseguito un confronto tra dati misurati sul campo e quelli ottenuti da criteri ben noti di letteratura allo scopo di definire la migliore equazione utilizzabile a scala regionale per quantificare l'apporto solido fluviale alle spiagge romagnole considerando uno scenario a medio termine.

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1. Introduction

1.1 Sediment production

Draining water from higher to lower elevations, rivers are the main natural network for sediment production and also principal routes of transport for the product of weathering from the continents to the sea. Sediment transport in rivers is fundamentally a two-phase problem in which water and solid sediment particles move through erosion and sedimentation processes at both long and short time scales (Schumm, 1977; Ariffin, 2016). Partitioned into suspended load and bedload, the sediment amount produced in the river basin through erosion from land surface by runoff and delivered to stream systems is defined as the sediment yield of a river catchment. The amount of the river sediment production is the result of complex hydroclimatic, hydraulic, and geomorphologic processes which occur on the river basin (Jansen and Painter, 1974; Ludwig and Probst, 1998). Sediment production capacity is strictly dependent on the supply of sediment from the catchment which is highly controlled by primary factors such as: climate, basin area, geology and topography. Soil texture, basin morphology as well as channel and bank erosion are further additional elements which also control river sediment loads resulting in variations over space and time in transport rate and concentration.

1.2 Human pressure on sediment yield

Sediment production is also influenced by land-use and land cover, human activities and presence of infrastructures, which, in a direct or indirect way, control the evolution of the entire catchment area manipulating geomorphic processes that shape river channels from big to small scale (Allan, 2004). In particular, it has been demonstrated that land use and land cover changes have a significant impact on both basin water cycles (Kondolf, 2002; Hooke, 2006; Aghsaei et al., 2020; Gaertner et al., 2019; Gumindoga et al., 2018; Cao et al ., 2009; Odongo et al., 2019;) and soil erosion dynamics (Smith et al., 2016). In fact, it has been studied that changes in land use have particular impact on evapotranspiration which, as an important component of the hydrological cycle, has important implications for the recycling of precipitation and generation of runoff, conditioning thus ecological, hydrologic and economic processes. Anthropogenic impact also includes activities such as water abstraction for irrigation, flow regulation, dam-and reservoir construction and river bed mining. Hydraulic works such as levees, dams, locks, reservoirs and weirs have always played an essential role in the development and utilization of water resources, in addition to protecting the population from flood risks. Despite this, the effects produced by their presence on the geomorphic responses of rivers can negatively affect the equilibrium of the fluvial environment (Kondolf, 1997;Ly, 1980; Liebault, 2001; Grant et al., 2003; Vörösmarty et al., 2003; Syvitski, 2005a; Graph, 2006; Hooke, 2006; Schmidt and Wilcock, 2008; Burke et al., 2009; Anthony, 2014; Pliquè et al., 2015; Martínez-Fernández et al., 2017). On this purpose, the continuous interruption of sediment transport equilibrium by engineering works or removal of sediment from the channel by gravel mining cause a reduction in sediment delivery from rivers to coastal areas (Kondolf, 1997). In addition to human alterations, an extra issue is also given by the impact of climate change. The reduction of the annual precipitation over the catchment basin is one of the main consequences (Billi and Fazzini, 2017). In fact recent studies are demonstrating how the combination of these factors (human alterations and climate change) are influencing hydrological processes. Globally, one of the main focuses of researches is the runoff which is an indispensable factor for water resource management (Huo et al., 2010;

Mwangi et al., 2016; Napoli et al., 2017; Khetrapal, 2018; Ayele et al., 2018). At this rate in the next future it is expected that climate change further exacerbates the situation on fluvial sediment loads of rivers (Ranasinghe et al., 2019).

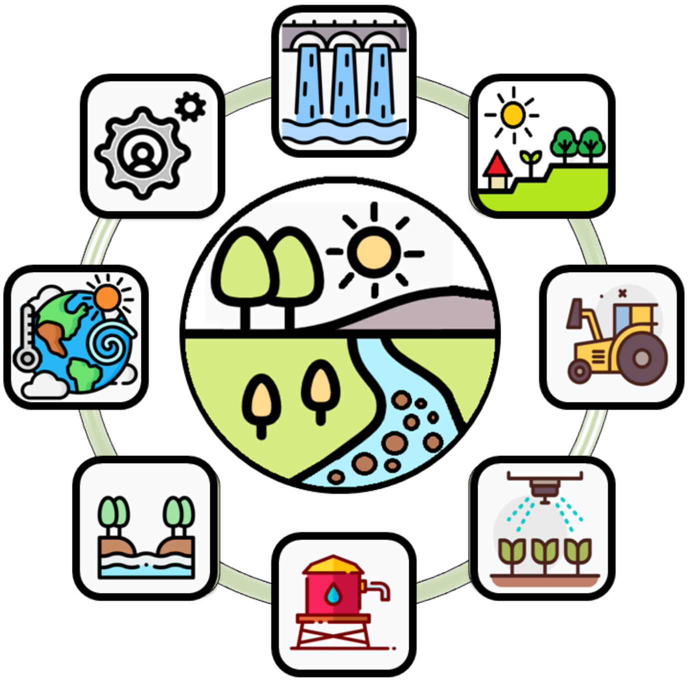

Figure 1.1 – Schematic overview of human impacts on fluviatile sediment transport. In the scheme with a clockwise orientation the following factors: hydraulic works such as dam construction; land use; gravel mining from riverbed; water abstraction for irrigation purposes; reservoirs construction; flow regulations for protection from flood risks; climate change; and other pressures.

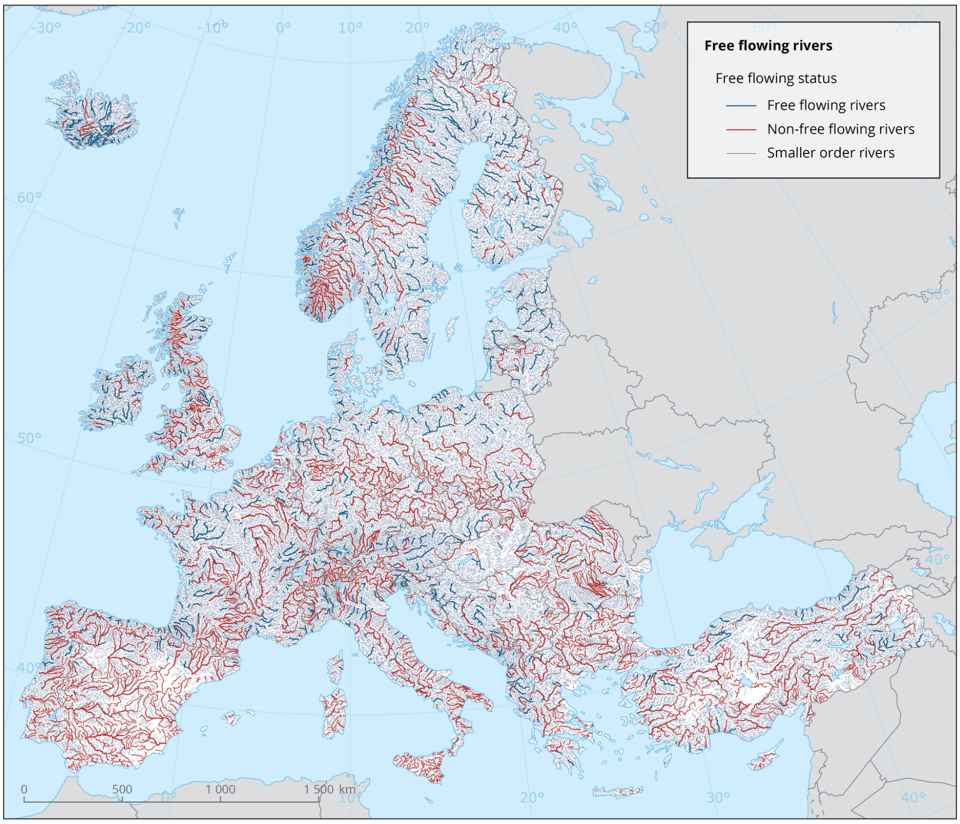

Figure 1.2 – Map of free flowing rivers in Europe (European Environment Agency (EEA)).

1.3 Importance of identifying and quantifying sediment yield

Understanding and quantifying sediment yield is thus becoming increasingly the center of attention of a variety of scientific and societal problems. The fields mainly involved are geomorphology, civil-and environmental engineering, sedimentary geology and river ecology which are all interconnected with each other (Sear et al., 1995). In fact the magnitude of sediment transported by rivers is important for the functioning of the entire fluvial system which includes: material flux, geochemical cycling, water quality, channel morphology, delta development, aquatic ecosystems and fluvial habitat quality. For example, sediment transport data are paramount to quantify sedimentation in reservoirs and to predict their life expectancy, to calculate the presence of risk areas or to correctly design river engineering works and irrigations schemes, as well as for sustainability of agricultural production. Furthermore, sediment plays an important role in the flux of several key elements, provoking changes in nutrient cycling or also resulting in pollution and habitat degradation in river systems. Sediment transport data are also essential to predict morphological variation of riverbeds and to determine whether they are natural or due to human intervention. Paramount importance of sediment yield is also related to coastal stability since rivers are the main source of sediment to coasts contributing to their formation, stability, preservation and management. Hence the importance and urgency to understand and to measure river sediment transport which has expanded considerably in the last three decades at both scientific research and professional level (Newson, 1995). Information about sediment yield, and bedload yield in particular, is very limited worldwide (Erskine and Saynor, 2015). Moreover often bedload component is also ignored or eventually estimated as fixed fraction of the total sediment load (Powell et al., 1996; Turowski et al., 2010; Ziegler et al., 2014). Several are the factors which may explain this lack of data. The main reason is linked to the feasibility of the measures, especially the ones performed in the field with direct techniques (Holmes, 2010). Technical difficulties, budget constraints, equipment availability, human resources, data requirement, efficiency strongly correlated with the transport rate variations and the characteristics of bed material are only some of the factors involved (Hubbel, 1987; Gomez, 1991; Gaeuman and Jacobson, 2006; Muhammad, 2019).

1.4 A focus on bedload transport on sand-bed rivers

The literature review revealed that considerably more attention has been given to bedload sampling in gravel-bed rivers than in sand-bed ones. Some of the plausible reasons why this happens could be that: in sand-bed rivers sediment transport occurs almost all the time, while in gravel-bed ones only in period of high floods; gravel-bed rivers are closer to sediment sources than sand-bed ones; in gravel-bed rivers bedload transport has an higher proportion of the total load than in sand bed rivers, where the suspended load is typically the major part of the total load; flow and sediment dynamic of gravel-bed rivers is less complex than sand-bed ones by the variety of bedforms occurring as a result of the interaction between the flow and the erodible bed (Bathurst, 1985; Holmes,2010). On this purpose, there are very few studies on field measurement of bedload in sand bed rivers and even more rare are the ones performed in coastal sand bed rivers close to their mouth (Kostaschuk et al., 1989; Morales et al., 2014). In fact measurement of bedload transport on sand –bed rivers has revealed to be a difficult task. Particular attention has been paid to sand-bed rivers since sand transport shows strong variabilities with time and space (Muhammad, 2019).The only studies on bed load transport addressed on sand bed rivers are referred to: Missisipi river, Missouri (Holmes, 2010; Abdel-Fattah et al., 2004), Missouri river, Missouri (Holmes, 2010); Quaresoo river, Iran (Haddachi et al., 2013); Nile river, Egypt (Abdel-Fattah et al., 2004); Rhine-Waal River, the Netherlands (Abdel-Fattah et al., 2004; Van Rijn 1991, 1992; Gaweesh and Van Rijn 1994); lower Fraser River, British Columbia (Martin and Ham, 2005). The scarcity of bedload field data highlighted by these studies underline also the short duration of the monitoring programs, except for a few cases (Martin and Ham, 2005).

1.5 A revision of bedload transport techniques

Several are the sampling devices and techniques used in the field to quantify bedload transport. Some of the principal methodologies used and described in scientific literature are: physical bedload traps installed in contact with the river bed (Laronne et al., 1992a, b; Reid et al., 1995; García et al., 2000; Bergman et al., 2007; Muhammad, 2019); the use of mechanical samplers (Helly and Smith, 1971; Sterling and Church, 2002; Vericat et al., 2006; Muhammad, 2019); the use of Acoustic Mapping Velocimetry, AMV (Holmes, 2010; Muste et al., 2016) and, finally, the use of Virtual Velocity of the bed-material sediments through Acoustic Doppler Current Profiler, ADCP (Rennie et al., 2002; Kostaschuk et al., 2005;Villard et al., 2005; Holmes, 2010). Both bedload traps and mechanical samplers necessitate time-consuming field campaign to perform in the context of short term projects such as the ones on river engineering (Vazqez-Tarrio and Mendez-Duarte, 2015). Furthermore they require prior knowledge of flow velocity and depth which implicates even more time consumption(Muhammad, 2019). Additional problems are linked with the disturbance that the flow and bedload transport could have in placing the samplers directly on the streambed (Holmes, 2010). Concerning the AMV use, despite it has proven to be a good method, it works better in larger rivers than in smaller ones. Limitations are linked with the necessity of sufficient time during which the flow stays steady to perform a successive bathymetric survey at the same flow discharge (Holmes, 2010). Finally, even bottom track approach (ADCP) necessitate more research, in particular more accuracy in determining the active depth of the streambed sediment movement is required to proper measure bedload transport (Holmes, 2010). Thus, despite continued efforts made to achieve increasingly accurate and advanced methods for bedload transport measurements, bedload has proven to be one of the most difficult fluvial processes to measure. Field measurements are very useful since they permit to obtain reliable data which can be used for model validation and application (Marquis and Roy, 2012). Bedload field measurement and its correlated difficulties and high costs have pushed scientists to develop equations for bedload transport prediction (Schoklitsch, 1934; Bagnold, 1980). Various formulas on bedload transport have been proposed in the past fifty years based on both laboratory and field studies (Habersack and Laronne, 2002), though field measurements are very limited (Molinas and Wu, 2001). One big limitation of bedload transport equations is that they return reliable results only under the specific conditions in which they were developed. Some formulas, in fact, are more appropriate to one type of channel morphology than others (Sidari et al., 2014), requiring a subsequent adaptation of the original formulations to the new condition (Khorram and Ergil, 2010; Haddachi et al., 2013) which is not always returning reliable results. The capability and accuracy of these formulas in returning bedload rates at an acceptable level are still questionable and the quality of their performance depends much on the specific river environments (Stevens and Yang, 1989; Habibi, 1994; Recking, 2010). Very few are the studies in the literature about the validation of bedload formulas with field data (e.g. Gomez and Church, 1989; Abdel-Fattah et al., 2004; Martin and Ham, 2005; Gao, 2012). Most of the criteria reported in literature largely rely on data from laboratory experiments where prototype conditions are performed (Molinas and Wu, 2001; Abdel-

Fattah et al., 2004). This creates reasonable limitations in their use for bedload estimation especially in relation to a component very difficult to be determined in bedload studies that is the upstream sediment supply. In fact difference in flow depth, Reynolds number, Froude number and water surface slope are only some of the elements which demonstrate significant difference between field and laboratory flumes (Molinas and Wu, 2001).

1.6 The role of small river catchments of the Mediterranean Sea and their relevance for future perspective for sediment supply

The interest in tackling all the above mentioned scientific problems has developed considering small river catchments. In fact, they can be considered more amenable to bedload transport studies especially considering them from a logistic (and practical) point of view. In particular the analysis of the contribution of runoff and the alteration of streamflow regime along the drainage network of a small basin is much faster than in a wider river system as well as the timing of water delivery. Furthermore the understanding of the discharge magnitude and the regime of flow with their rapid variations consequently permit major practical operability in field campaigns. Therefore the role of small rivers has been recognized as important in contributing to the global sediment budget (Milliman and Syvitski, 1992; Inman and Jenkins, 1999). Information given by the analysis performed by Milliman and Syvitski (1992) demonstrated how basin area and maximum elevation of the river basin control both water and sediment discharges: results of their research demonstrated how small river basins have higher runoff than larger ones and consequently higher sediment yield.
Hence the importance on small steep-gradient rivers in the Mediterranean Sea and their overarching relevance in sediment supply for future perspective associated with many purposes. One of these, and of primary interest, is connected with the protection and the maintenance of the coastal environment which in the recent years has undergone alterations. Changes in sediment supplied by Mediterranean rivers together with climatic changes and sea-level rise have impacted upon the Mediterranean coastal zones (Poulos and Collins, 2002). Continuous anthropogenic influence on natural processes has contributed to reducing the overall fluviatile water-sediment supply to the Mediterranean basin, influencing both beach formation and stability (Poulos and Collins, 2002).

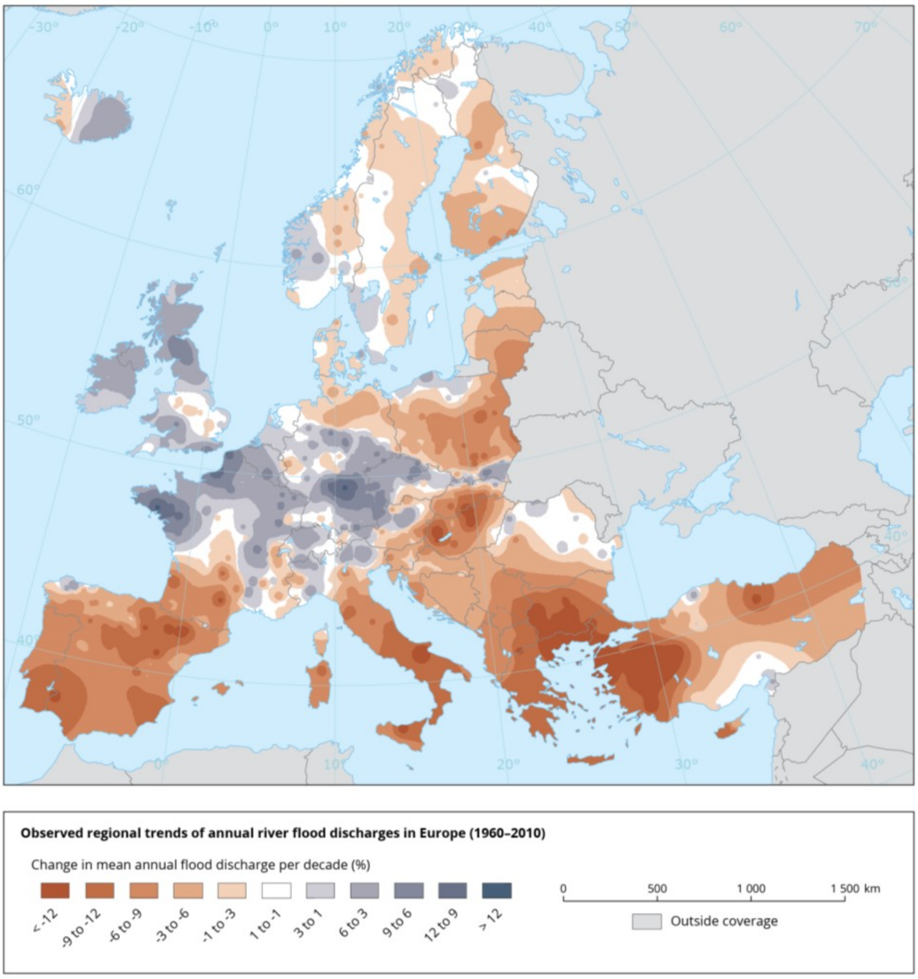

Figure 1.3 – European river flood discharges observed in the past 50 years (European Environment Agency (EEA)).

1.7 Fluviatile sediment supply connected to coastal erosion processes in Italy: the situation in the Emilia-Romagna region

One of the most effected nations is Italy whose coasts have been subjected to alarming coastal erosion processes (Fierro, 2004; Cantasano et al., 2017). Unfortunately, in Italy, the field data of sediment transport, and especially of bedload, are very scarce and the monitoring campaigns were relatively short (Billi and Paris, 2002, 2004, 2014; Francalanci et al., 2013; Francalanci et al., 2015). The coast of Emilia-Romagna region is one of the most affected by beach erosion of the Adriatic Sea (Cantasano et al., 2017). In this region beach preservation is crucial for tourism activities which are an important part of the local economy (Armaroli et al., 2006; Armaroli et al., 2012; Perini et al., 2016). Tourism and recreational activities are, in fact, very common in the summer months. In this sector of the Adriatic coast, beach retreat started a few decades ago and has worsened in the last years. Healthy beaches are important to prevent the inundation of coastal villages during storms, especially in this low-lying territory and in association with the expected climate changes and sea level rise. Expensive coastal protection countermeasures were deployed to decrease inshore wave height (i.e. hard structures) or to refill sectors in erosion (i.e. beach nourishments). Unfortunately, the efficiency of the structures is questionable and nourishments have to be repeated every few years to counteract the scarcity of natural sediment supply from rivers. The sediment yield of the Emilia-Romagna rivers is still undefined, except for a couple of cases which refer to Fiumi Uniti, Reno and Bevano rivers

(Billi and Salemi 2004; Billi et al., 2017; Ciavola et al., 2005; Ciavola et al., 2010; Preciso et al., 2011).

Poor knowledge of regional river supplies is the reason why this study has been developed. It is based on field measurements and investigations on two representative rivers with the goal to widen the actual dataset of the supply of sand from rivers at regional scale. By means of hydraulic and hydrological approach and, most of all, by means of bed load transport measurement campaigns in the field, this research aims at defining the quantity of sediment supply in the study rivers, and to test existing prediction models or to develop new ones to be applied to the whole Romagna coast. The fluviatile sediment input to the coastal budget has never been estimated despite its importance. Due to intrinsic technical difficulties in the field measurements, previous attempts of bedload yield assessment were based mainly on theoretical calculations without any field data validation.

1.8 Notes of the Emilia-Romagna catchments

Within this context, the herein doctoral thesis aims to investigate the sediment dynamics in two small river systems (Fiumi Uniti and Savio), both considered pragmatic in terms of sediment contribution for the Emilia-Romagna coastal sediment budget alteration. Originating from the northern Apennines, they both outflow in the Adriatic Sea near Ravenna province. With a total catchment basin of about 1000 km2 (Fiumi Uniti) and 647 km2 (Savio), the two river basins can be considered essentially similar since they have analogous characteristics. Flanking to each other, the Fiumi Uniti and the Savio basins are located in the mid to southeastern area of the Emilia-Romagna region. Bordering to the north on the Lamone basin, the Fiumi Uniti is flanked by the Bevano and Savio basins on the south which, in turn, is flanked by the Marecchia basin (south) briefly touching also the Rubicone and the Uso basins on its south-east part. The Savio basin is not entirely located in the Emilia-Romagna region: in fact one small part of the basin (12%) is located in the Marche region. Delimited by the Apennine watershed almost entirely coinciding with the regional border, the Fiumi Uniti basin constitutes the most important hydrographic system of the southern part of the region. It is made up of two main rivers, Ronco and Montone, which join each other in Ravenna. Originally they both outflowed in the Adriatic Sea. The junction between Montone and Ronco was realized by the citizens of Ravenna around the 1700s in order to avoid repeated floods in the city. With a total reach length of almost 90 km, the Montone river originates near the Muraglione Pass (836 m a.s.l.). The Rabbi, tributary of the Montone, originates near Monte Falco and flows into the Montone river near Forlì, with a reach length of almost 56 km. The Ronco river is formed by the union of three branches which originate respectively in Corniolo (1400 m., a.s.l.), Ridracoli (1200 m a.s.l.) and Strabatenza (1200 m a.s.l.). It has also a tributary, called VoltreI, wich originates in Monte Calbano at 650 m a.s.l. and flows into the Ronco at Meldola. The total length of the Ronco is about 135 km. (80) With a total length of 126 km, the Savio river originates near Monte Castelvecchio (1060 m a.s.l.). While the mountain basins of the Montone and Ronco rivers end at Forlì, the Savio basin closure is located at Cesena. The Borello torrent, tributary of the Savio, originates in Monte Aiola (942 m a.s.l.) and, after 26 km, flows into the Savio at the village of Borello. Differently from the mountain side, all rivers (Montone, Ronco and Savio) are largely dammed in the plain area. The geological characteristics of the territory constituting the basins are different in the various sections. Two-thirds of the basin surface are located in the northern Apennines (maximum elevation 1650 m) and are underlain by Miocene turbidites consisting of sandstones and marlstones alternations (Amorosi et al, 2002). The lower portions of the catchments are underlain by Pliocene marine deposits and the Quaternary Po river alluvial deposits (Amorosi et al, 2002). Perennial springs in the basins that feed the waterways are only a few, therefore the outflow is considerably affected by precipitations. The climate is conditioned by its geographical position in the center of the northern temperate zone and on the southern edge of the Po Valley between the Apennine ridge and the Adriatic. Thus affected by climatic characteristics of the Po Valley. The Adriatic Sea, closed and shallow, has a moderate mitigating influence on the climate. The climate is subcontinental temperate (Liebault and Piegay, 2001) with a mean temperature averaged over the 1961-2010 period of 13-13.5°C for most of the basins (Antolini, 2016). Mean annual cumulated precipitation, averaged over the same period, varies from 600 mm in the coastal areas to 1800 mm in the catchment headwaters (Antolini, 2016). The climate is characterized by dry summers and precipitation peaks during the winter and spring season. Rainfall is usually greatest in spring period and involves the months of March, April and sometimes extend until May. Precipitation occurs mainly as rainfall, and to a small extent in the upper portion of the basins, as snowfall. Both catchments are characterized by the presence of hydraulic structures which are present both in the mountain part of the basins and in the plain area. Among the most important ones there is the Ridracoli dam. With its height of 103.5 m and length of 432 meters, the dam forms a lake of 100 hectares and an altitude of 557 m asl. The huge invaded, capable of 33 million cubic meters, is able to quench the whole south part of the Emilia-Romagna region. The principal hydraulic structure present in the Savio river is the Quarto dam, an artificial dam built over the natural dam for hydroelectric purposes. The lower portions of the catchments are regulated by sluice gate dams, principally used for agricultural purposes. Among these, there are the San Marco on the Montone and San Bartolo on the Ronco. On the canalized Fiumi Uniti the Rasponi dam, located 3.5 km upstream of the river outlet, is closed in summer to prevent saltwater intrusion. All dams regulated by the local Water Authority are kept open during flood season (October-March) allowing the river sediment to reach the coast. In the rest of the year the sluice gates are permanently closed to ensure water retention for agriculture. In the lowest part of the Savio reach there is the Castiglione dam which, as the ones on the Fiumi Uniti, is used for water retention.

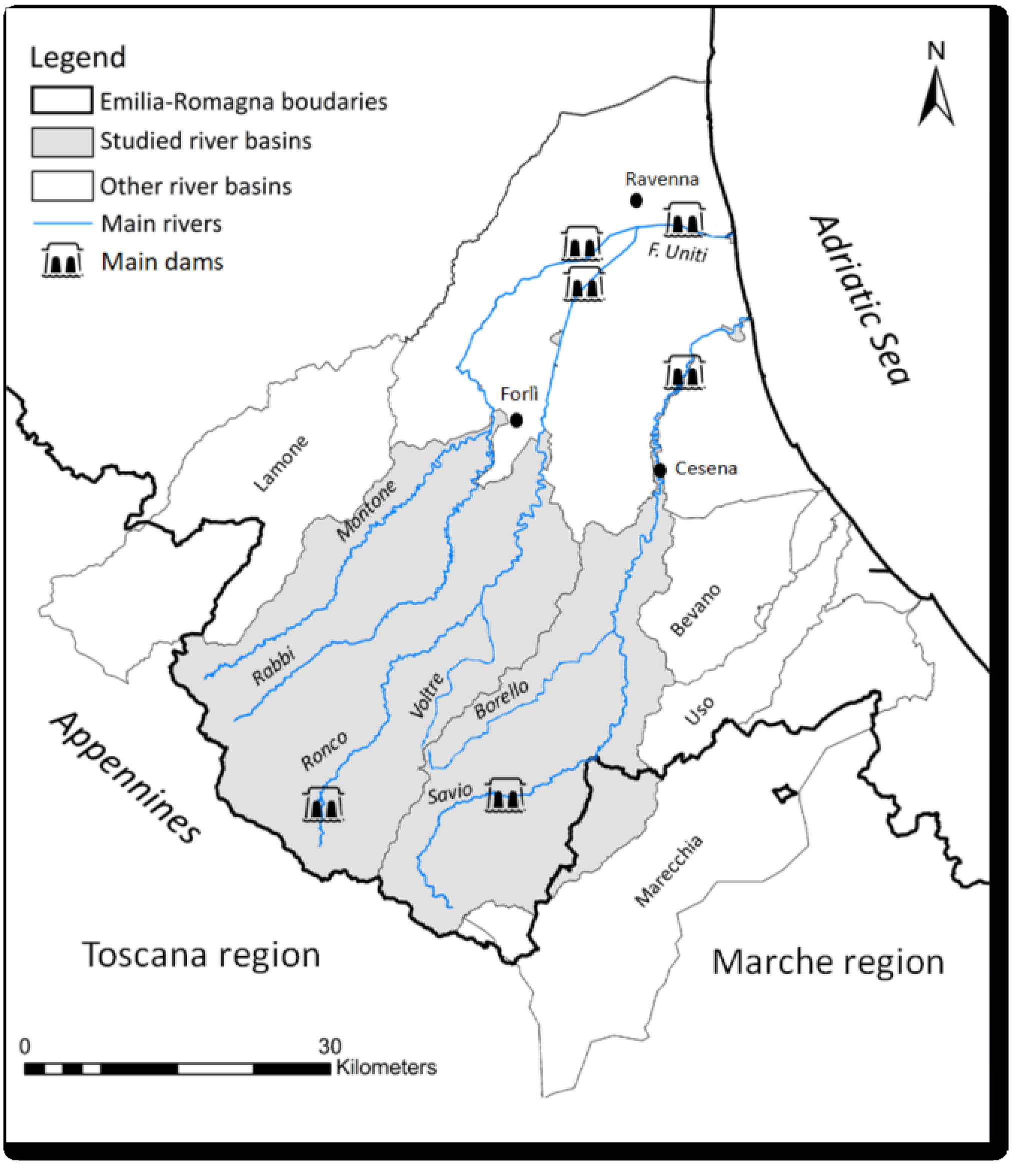

Figure 1.4 – Location of the Fiumi Uniti and Savio river basin

1.9 Aims of the project

Given the poor knowledge of river supplies at regional scale, this PhD research aims at quantifying the bedload yield to the Romagna beaches through field investigations and measurements on two representative rivers to enlarge the actual dataset. The field data are then used to define a methodological criterion applicable at a regional scale, able to quantify the sediment flux also of the other rivers of the Emilia-Romagna region. An analysis of bed-load transport rates for the last 10 kilometers of the river has been done. Multidisciplinary analysis which includes hydraulic investigation and modeling of the river reaches just shortly upstream of their mouths have been performed supported principally by direct field measurements (Helley-Smith bedload sampler). Started previously (in 2005-06) by Billi at al. (2017), field campaigns have been completed in 2019 by the author of this thesis with an additional three year campaign on the Fiumi Uniti and a completely new dataset measured on the Savio. A significant number of field measurements were made resulting in a total dataset which amounts to 24 and 14 floods on the Fiumi Uniti and Savio, respectively. Repeated bed-material samplings were carried out during different flood conditions have permitted to calculate bedload rating curves (and annual bedload yields). Low and high flood events were measured giving the possibility to observe the threshold conditions of incipient bedload motion in the field and to know its value. Additional bathymetric surveys of the river downstream reaches were carried out, revealing the presence of bedforms which have been also investigated since they play an important role in the interaction of sediment transport and flow processes. The field measurements allowed to identify with great accuracy the field condition for the threshold of bed particle movement. This circumstance proved to be very useful to test also equations to determine the hydraulic conditions for bed particle entrainment. The correct determination of the threshold flow is a very important issue since many of the bedload equations that are used to predict the bedload yield of coastal river include also the threshold condition as a reference datum for the calculation of bedload. Given that in several equations the threshold condition is expressed in terms of shear stress, also the occurrence of the roughness effect of dune bedforms has to be taken into consideration. Important components affecting the bedload transport rates and its calculation using the classical bedload equations were also investigated. The field measurements of this study provide a unique chance to test some of the most renowned bedload equations against field data and to select those which are more appropriate for the rivers of the Adriatic coast. Finally, an attempt to define the best equation to be used at regional scale to quantify bedload supply to beaches in a mid-term scenario has been performed.

1.10 About this PhD thesis

This doctoral thesis follows the ‘thesis by publication’ format. All the articles presented in this collection are both in press or under review in scientific journals, in proceedings of conferences, or books. All contributions are the result of work which belongs to COSTUF (coastal geomorphology research team of the Department of Physics and Earth Sciences of the University of Ferrara, Ferrara, Italy) with contributions from Professor Paolo Billi (International Platform for Dryland Research and Education, Arid Land Research Center, Tottori University, Japan) and Professor Leonardo Schippa (Engineering Department, University of Ferrara, Ferrara, Italy). The PhD candidate played a dominant role in carrying out most of the field activities, collecting and analyzing data as well as interpreting the outcomes and writing large part of the papers.

The main body of the manuscripts present and discuss the most important research issues including the analysis of fluid and solid discharges, the examination of the threshold condition of sediment motion, the presence and the role of bedforms on flow resistance and finally the construction of a predictive model able to quantify bedload supply of the study rivers as a fundamental component to understand the sediment budget of the beaches subjected to severe erosion in the coastal zone of Romagna. The final chapter contains a conclusive summary of the main outcomes of the PhD thesis. A detailed short description of each chapter follows:

- Chapter 2 – this work is based on some preliminary field data of water and bedload discharge measured in the Fiumi Uniti river. In this paper, field data were used to address the topic of the mathematical representation of the bedload motion and the relative threshold condition for sediment motion initiation.

  The manuscript is in publication in the book Mathematical Approach to Climate Change and its Impacts as:

  Cilli S., Billi P., Schippa L., Grottoli E., Ciavola P. In press. Bedload transport processes in a coastal sandbed river: the study case of Fiumi Uniti river in the northern Adriatic. Mathematical Approach to Climate Change Impacts. Springer INdAM Series, 38 (Rome, Italy). doi: 10.1007/978-3-030-38669-6, 2020.

- Chapter 3 – this work is an extension of the previous one. It examines the threshold conditions of bedload transport considering two rivers, Fiumi Uniti and Savio. An estimation of critical shear stress values compared to the field data is performed following three different approaches: Shields's criterion (revised by Brownlie and Simoes), Carling and Hammond's and Bagnold approach.
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  The manuscript is published in the Proceedings of River Flow 2018 (Lyon-Villeurbanne, France, September 5-8, 2018), E3S Web of Conferences as:

  Cilli S., Billi P., Schippa L., Grottoli E., Ciavola P. 2018. Field data and regional modeling of sediment supply to Emilia-Romagna's river mouths. E3S Web of Conferences 40, 04002. https://doi.org/10.1051/e3sconf/20184004002

- Chapter 4 – this paper focuses on flow resistance in sand bed alluvial rivers, paying particular attention on the contribution of moving dunes. This study proposes a model tested on a large dataset, including also the Fiumi Uniti and Savio rivers.

  The manuscript is published in Water as:

  Schippa, L., Cilli, S., Ciavola, P., Billi, P. 2019. Dune Contribution to Flow Resistance in Alluvial Rivers. Water, 11, 2094. https://doi.org/10.3390/w11102094.

- Chapter 5 – this paper is the product of field measurements and investigations aiming at examining the interaction between bedload transport and dune morphology. This study explores the fundamental mechanisms of flow resistance, which results from grain and bedform roughness, by comparing two models available in literature, i.e. Engelund and Hansen (1967) and Engelund and Fredsoe (1982), with the one proposed in Chapter 4. This paper is based on field measurements campaigns, hydrodynamic modeling and bathymetric survey.

  The manuscript has been recently submitted as:

  Cilli S., Billi P., Grottoli E., Schippa L., Ciavola P. Submitted. Moving dunes constrain flow hydraulics in mobile sand-bed streams: The Fiumi Uniti and Savio River cases (Italy). Geomorphology.

- Chapter 6 – This paper summarizes the relation between water discharges and bedload discharges in the study rivers. The work includes also a comparison of the field results with renown formulae available in literature. The main aim of this paper is the reckoning of mid-term water and sold discharges data.

  The manuscript will be submitted in the next weeks as:

  Cilli S., Billi P., Schippa L., Grottoli E., Ciavola P. In preparation. Evaluation and mid-term reconstruction of bedload transport of two small sand-bed rivers of the Emilia-Romagna region (Italy) through direct measurements.

The conclusive chapter (Chapter 7) is a summary of the findings of each chapter. It gives an overview of the study addressed. It includes answers to the main research questions faced before and during this PhD study.
The PhD thesis is addressed to students, researchers, and professionals that work with fluvial environment. It would be also useful to fluvial (and coastal) managers, especially of the Emilia-Romagna region, in order to establish the role of its rivers on the coasts. Furthermore, the data obtained would enlarge not only the existing Italian dataset but also the international one. In fact worldwide these type of measurements are scarce and only performed for short time periods which consist of not more than one or two years. In addition, many of these measures are made by government agencies for specific project and not published on the international circuit.

2. Bedload transport processes in a coastal sand-bed river: the study case of Fiumi Uniti river in the northern Adriatic

2.1Preamble

Mechanics of sediment motion in open channels has been investigated for more than 150 years. Nevertheless, methods developed to predict the threshold conditions of sediment transport and the transport rate have been unsatisfactory for long time (Simons and Senturk, 1992). Little progress has been made over the last three decades.
First of all, we have to define what initiation of motion means. Several are the definitions available in the literature (Neill and Yalin, 1969; White, 1970; Dancey at al., 2002), but in general it can be concisely defined as the condition for which particles lying on a streambed start moving under fluid action. Therefore, the shear stress needed to initiate bed particle motion is defined as that stress which produces an appreciable amount of bedload transport. Threshold entrainment condition is really difficult to define in the field and, due to this limit, several empirical threshold curves have been developed based on concept of mobility number, i.e. that number which represent the dimensionless balance between disturbing and stabilizing forces on a sediment particle under flow: moments of the fluid forces of drag and lift with the resisting moment of the submerged particle weight (Leliavsky 1966; Helley, 1969).
Initial movement of loose grains over a surface has been pioneering studied by Shields in 1936 and further re-proposed through empirical threshold curves. The threshold state which has been notably expressed by Shields (1936) describes individual particles on a sedimentary bed of nearly spherical shape and uniform size in motion by a unidirectional flow. It relates the particle Reynolds number $Re_* = (u_* d)/\nu$ to the dimensionless critical shear stress (θ_{cr}) defined as Equation 2.1:

$$\theta_{cr} = \tau_{cr}/[(\rho_s - \rho)gd] \qquad (2.1)$$

Where τ_{cr} is the critical bottom shear stress in kg/ms^2, ρ_s and ρ are the sediment and fluid densities in kg/m^3, g the gravity acceleration in m/s^2 and d the particle diameter in m. Moreover $u_*$is the critical shear velocity for incipient motion in m/s and ν the fluid cinematic viscosity of the water in m^2/s.
Shields (1936) demonstrated that θ_{cr} of near-uniform grains varies with Re_*, hypothesizing that θ_{cr} attains a constant value of about 0.06 above Re_* 489; Komar in 1988 revised this value approximating it to 0.045 for gravel particles, lower than the original Shields value. At lower values of Re_* the function reach a minimum of $\theta_{cr} = 0.03$.
The use of critical shear stress τ_{cr} and critical shear velocity u_* (defined as $\sqrt{\tau/\rho}$) on both axes required an iterative procedure. In order to avoid errors in the iterative procedure, some researcher proposed a direct computation of the critical shear velocity u_* through the use of a dimensionless grain diameter D_* defined as Equation 2.2:

$$D_* = \left[\ (\rho_s - \rho)/\,\rho\left(\frac{g}{\nu^2}\right)\right]^{1/3} d \qquad (2.2)$$

and commonly used in the threshold curves (Van Rijn, 1993).
As evidenced by Beheshti and Ataie-Ashtiani (2008), a wide review of the original Shields (1936) curve based on this rewriting has been done and it is present in literature as subparallel Shields curves (Bonfille, 1963; Brownlie 1981; Chien and Wan, 1983; Cheng

and Chiew, 1999; Paphitis, 2001; Hager and Oliveto, 2002;Cheng, 2004; Cao et al., 2006; Simoes, 2014). Moreover, several additions, revisions, and modifications of the original Shields (1936) curve have been done recognizing also the importance of intergranular geometry of the bed material (i.e., friction angles), as well as irregularly grain shape, sorting, and packing (Miller and Byrne, 1966; Li and Komar, 1986; Kirchner et al., 1990; Buffington et al., 1992).
In this section, the Shields (1936) revised criteria and also a simpler model based on empirical power law relationship have been computed for the studied rivers. To determine the initiation of motion from field measurements (complementarily with hydraulic modeling) at a range of flow which produces appreciable and measurable bedload transport is the first step in the revision of suitable criteria to predict the rivers sand supply to beaches at regional scale.

2.2 Introduction

Rivers are the main natural network for the sediment transfer to beaches, contributing to their formation and stability. The amount of sediment that is released at their mouths is the result of complex hydrological and hydraulic phenomena. Yet, it depends on the geomorphological dynamics of the river basin, but it is also influenced by human activities and infrastructures, which, in a direct or indirect way, control the evolution of the catchment as well as the marine-coastal area. Anthropogenic impacts, such as changes in land use (mainly increase in the forested land), river bed mining, presence of engineering works and progressive dismantling of the rivers mouths, are only some of them (Kondolf, 1997; ; Liebault and Piegay, 2001; Grant et al., 2003; Hooke, 2006; Anthony, 2014). For all these reasons, in the last decades, many beaches, particularly in Italy and other industrialized countries, have been affected by marked erosion that is still progressing. The Emilia-Romagna is an Italian region particularly affected by this phenomenon, which is mainly due to the reduction of sediment supply by the local rivers. As well as for larger fluvial systems, the role of small rivers has been recognized as important in contributing to both beach stability and changes (Inman and Jenkins, 1999). Unfortunately, despite few sporadic field studies, bed-load sediment transport in the Emilia-Romagna region is poorly known and referred to a limited number of rivers (Billi et al., 2004; Ciavola et al., 2005; Ciavola et al., 2010; Billi et al., 2017). In order to widen the data set of river sediment supply in the whole region, bedload measurement campaigns in representative rivers are in progress by the authors of the current paper. As part of a regional scale project, which includes hydrological investigation and bed load transport monitoring, the Fiumi Uniti study is a pilot study aiming to define the sediment supply to the whole Romagna coast. Main points of this project are a review of existing bedload field data and analysis of additional data in order to qualify and quantify the sediment budget of the Fiumi Uniti river, a typical example of a small river system in the Emilia Romagna region. The research activities include hydrological investigations and bed load transport measurements about 8 km upstream of the river mouth.

2.3 Study area

The Fiumi Uniti catchment is about 1000 km^2 and derives from the unification of two rivers: the Montone and Ronco. Both rivers have similar morphometric characteristics (Table 2.1). They flow from the Apennine Mountains to the Padan plain, where they merge near Ravenna. The river mouth is located between Lido Adriano e Lido di Dante, on the Adriatic Sea (Figure 2.1).The Fiumi Uniti is an artificial river 9.3 km long (from the

confluence to the sea) that was realized by the citizen of Ravenna around the 1700s in order to connect the Montone and Ronco rivers to the Adriatic Sea to avoid repeated floods in the city.
Two-thirds of the basin surface are located in the northern Apennines (maximum elevation 1650 m) and are underlain by Miocene turbidites consisting of sandstones and marlstones alternations (Amorosi et al, 2002). The lower portions of the catchments are underlain by Pliocene marine deposits and the Quaternary Po river alluvial deposits (Amorosi et al, 2002). The climate is subcontinental temperate (Liebault and Piegay, 2001) with a mean temperature averaged over the 1961-2010 period of 13-13.5°C for most of the basin and a mean annual cumulated precipitation, averaged over the same period, that varies from 600 mm in the coastal areas to 1800 mm (see Figure 2.2) in the catchment headwaters (Antolini, 2016) The Fiumi Uniti river is a typical example of a small river system, which is regulated by a sluice gate dam, located 3.5 km upstream of the river outlet. The sluice gate is closed in summer to prevent saltwater intrusion. However, during the other seasons and during floods, even the smallest ones, the sluice gate is kept open, allowing the river sediment to reach the beach. Recently, the local Water Authority decided to keep the sluice gate permanently open. At low flow the tidal influence is clearly observed in the data, though the tide range at the gate is evident, even though limited to about 0.4 m.

Parameters	**Units**	**Montone**	**Ronco**
Area	km^2	441.09	524.82
Perimeter	km	600.53	545.96
H_{max}	m asl	1245.17	1649.52
$H_{closure}$	m asl	30.03	24.84
H_{mean}	m asl	450.36	484.94
S_{mean}	%	39.06	39.38
$L_{main\ reach}$	km	90.19	92.65
R_c	-	0.008	0.011
R_f	-	0.34	0.38
R_{el}	-	0.26	0.28
C_{comp}	-	25.33	21.11
F_f	-	0.05	0.06
LPD	km	58.00	68.89
DD	Km km^2	0.13	0.13

Table 2.1-Morphometric parameters of the Montone and Ronco rivers. The listed parameters are: area, perimeter, maximum altitude (H_{max}), altitude at the closure of the basin ($H_{closure}$), mean altitude (H_{mean}),mean slope of the basin (S_{mean}); circularity ratio (R_c), basin shape ratio (R_f), elongation ratio (R_{el}), compactness coefficient (C_{comp}), form factor (F_f), longest drainage path (LPD) and drainage density (DD).

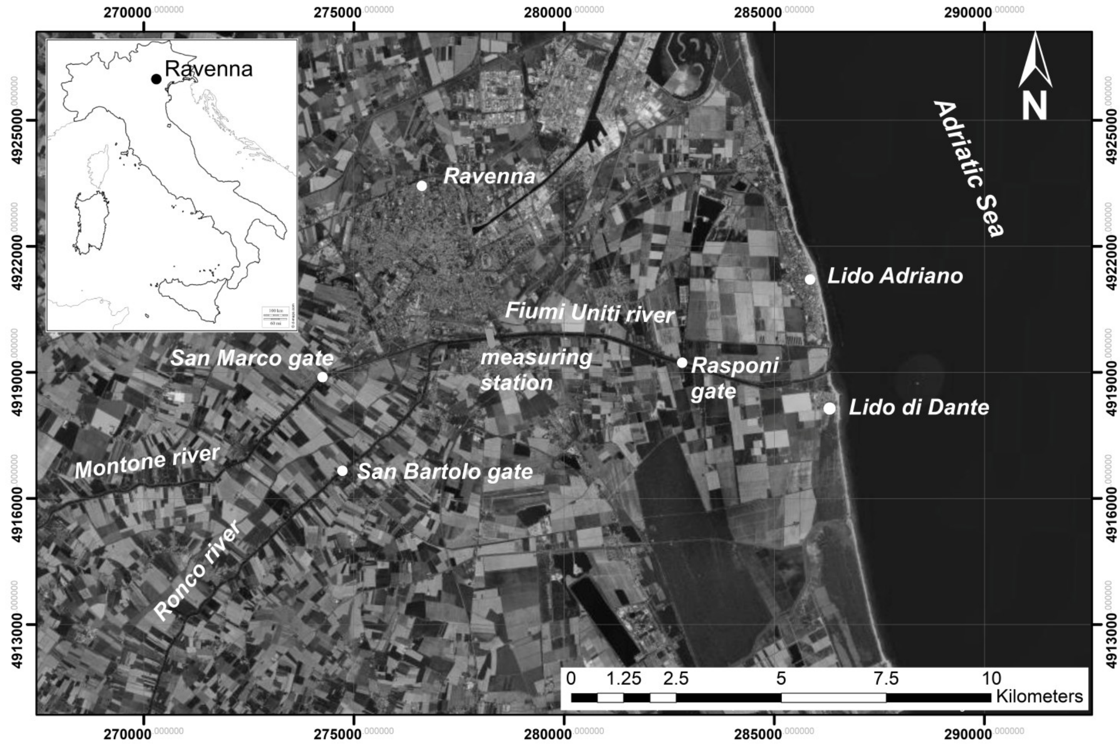


Figure 2.1-The Fiumi Uniti river with measuring site and sluice gates.

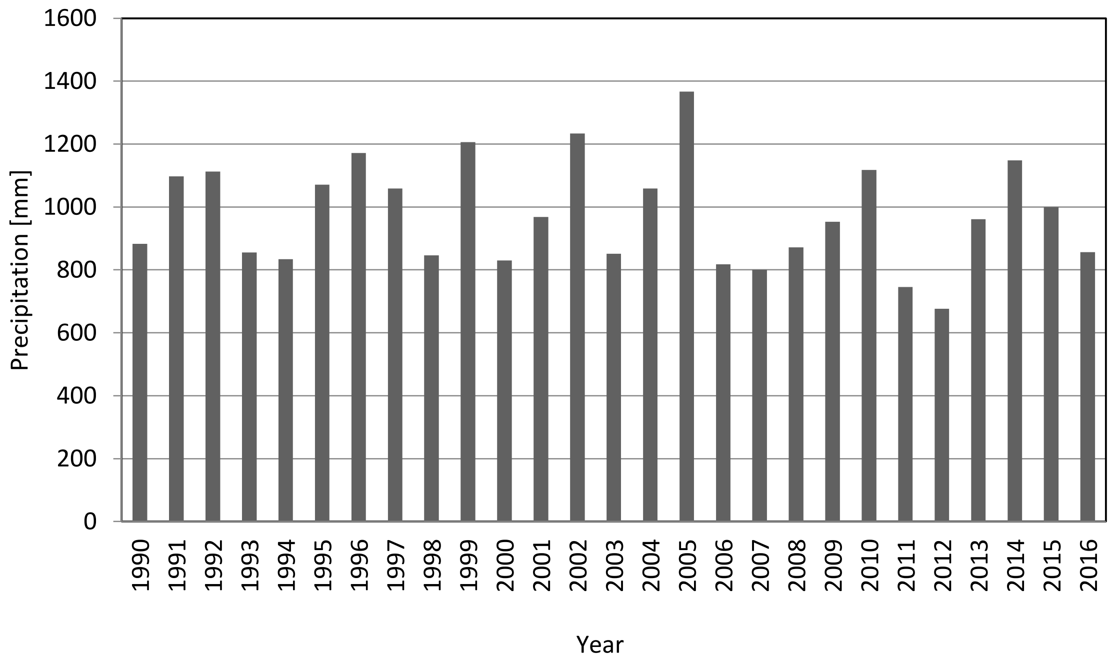


Figure 2.2-- Annual precipitation on the Fiumi Uniti river basin in the 1996-2016 interval, data obtained from the Italian Hydrographic Service.

The Montone and Ronco rivers have sluice gates too, located a few kilometers upstream the river junction. These gates are for irrigation purpose, and they are regulated according to the seasonal rainfall conditions: they are open during the flood season (October-March) and closed during the dry ones (April-September). The monitoring station is located in Ravenna (Figure 2.1) at a pedestrian bridge. In this reach the river shows a prismatic channel (Figure 2.3) having a rectangular cross-section (Figure 2.3) and a streambed gradient of 0.00029 m/m (Billi et al., 2017). The maximum channel width at the levee crest level is about 60 m and the active channel width, i.e where bed load transport occurs, is 40 m. The bed material is predominantly sandy, with an averaged D_{50} of 0.43 mm in the active channel width (Billi et al., 2017).

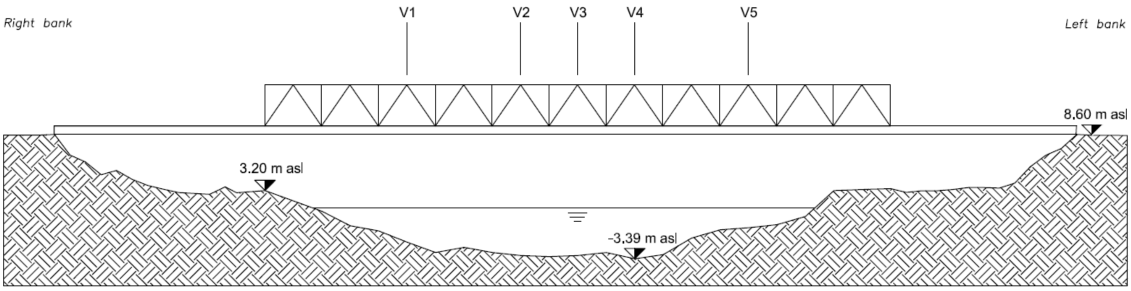


Figure 2.3- The cross-section of the river. The figure shows the position of the 5 verticals used for the monitoring activities. The bankfull level can be approximately fixed around 3 m above mean sea level, i.e. at the level of the first bank (3.20 m asl), as illustrated. The figure shows also the altitude of the stream bed at the present cross section (thalweg equal to -3.39 m asl) and the one of the main bank (8.60 m asl). The values reported are referred to the cross section condition before the monitoring activities.

2.4 Methods and instrumentation

The analysis of sediment transport is based only on data obtained by in situ measurements at the monitoring station. Part of them comes from 2005 and 2006 field measurements by Billi et al. (2017). Additional data were obtained from field measurement during two floods occurred on 07/02/2017 and 07/03/2017. During these floods, hydraulic and sediment monitoring was carried out at five verticals, equally spaced across the active channel width. At each vertical flow depth, flow velocity and bedload transport were measured. Flow velocity was measured with a standard USGS AA type current meter, with vertical axis. Bedload transport was sampled with a standard Helley-Smith bedload sampler (US BL-84) with a 76x76 mm intake and an expansion rate of 1.10, which is considered to provide the highest efficiency (Emmet, 1979). The sample bag of the Helley Smith had a 0.1 mm of mesh. All the instruments were lowered from the pedestrian bridge with the help of a wheel crane. Measurements were taken at established time intervals according to the water level rate of change detected by a staff gauge, installed on the left bank of the cross section. The measurements across the whole cross-section required about one hour. As pointed out by Emmet (Emmet, 1979), the sampling time of the Helley-Smith sampler is not standardized and has to be calibrated after a few attempts, given the high variability of bedload transport during different phases of a flood. All the sediment samples collected

were cleared from vegetation debris and organic material. Each sample was wet sieved to remove the fine fraction (finer than 63µm. The coarser fraction was dried for one day at a temperature of 105 °C. Then each sample was dry-sieved for 20 minutes with a standard Ro-Tap shaker with 0.5 phi scale sieves. The sample portion finer than 63 mm was not included in the calculation of bedload since that material is part of the suspended load that was accidentally collected by the sampler bag, together with vegetation debris.

2.5 Mathematical and statistical approach

Bed-load sediment transport by water flow in natural streams depends on bed material characteristics and flow condition. Usually, classical bed-load formulas available in the literature define the sediments transport as the continuous contacts of the particles with the bed strictly limited by the effect of the gravity. In fact the mathematical representation of the bed-load transport motion is mainly distinguished in: rolling, sliding and saltation. The reach of study shows a regular geometry, and the rate of change of flow (both in terms of water level and discharge) is relatively weak during a flood, therefore a quasi steady gradually varied flow has been assumed; and the shear stress results as in equation 2.3:

$$\boldsymbol{\tau} = \boldsymbol{\rho g R S} \qquad (2.3)$$

Where:

ρ is the density of the fluid [kg/m^3];
g is the gravity acceleration [m/s^2];
R the hydraulic radius [m], for wide and relatively shallow rivers R can be substituted by the mean flow depth H;
S is the energy gradient slope [m/m]. In the present study it has been calculated in two different ways: in the first case, it has been assumed equal to the stream bed slope, while in the second one it has been simulated through the Hec-Ras model.

When the value of the bed shear velocity (i.e. defined as $u_* = \sqrt{\tau/\rho}$) exceeds the threshold value for the initiation of motion, the particles will start moving constantly in contact with the bed. In the present analysis the transport is defined as previously explained. The criterion here used to predict the initiation of sediment motion is the Shields classical criterion (Shields, 1936) which is the most used in river dynamics and fluvial geomorphology. It introduces a dimensionless number of the critical shear stress θ_{cr} as shown in equation 2.4:

$$\theta_{cr} = \tau_{cr}/(\rho_s - \rho)gD \qquad (2.4)$$

Where:

τ_{cr} is the dimensional shear stress corresponding to the incipient motion[N/m^2];
ρ_s is the density of the sediment [kg/m^3];
ρ is the density of the fluid [kg/m^3];
g is the gravity acceleration [m/s^2];
D is the characteristic particle diameter of the sediment [m]. Generally the D_{50}, which is the median value of the particle size distribution, is used as the characteristic particle diameter of the sediment.

The Shields curve has been represented in analytic form by Brownlie (1981) as showed in equation 2.5.

$$\theta_{cr} = 0.22{R_p}^{-0.6} + 0.06e^{(-7.7{R_p}^{-0.6})} \quad (2.5)$$

Where R_p is equal to $(R^*((\rho_s/\rho) - 1\,))^{\,0.5}$ ·, R^* is the dimensionless Reynolds number $(u_* D/\nu)$, $u_* = \sqrt{(\tau/\rho)}$ [m/s] is the bed shear velocity, D the characteristic particle diameter of the sediment [m] and ν the cinematic viscosity [m^2/s].

2.6 Results and discussion

Eleven floods were monitored from 12 April 2005 to 7 March 2017. In this study only 9 of them are considered, i.e. those monitored with the sluice gates fully open. The weakest flood occurred on 11/10/2005 with a flow discharge of 17.27 m^3/s, whereas the largest one occurred on 12/04/2005 with 358.16 m^3/s, which can be considered one of the largest floods recorded in the last decade (Billi et al., 2017).
All the floods monitored are reported in Figure 2.4, where different flow rating curves are also shown. The data may be represented by the flow rating curve proposed by Herschy (1985) with the form $\mathrm{Q} = \mathrm{a}(\mathrm{x} - \mathrm{b})^{\mathrm{c}}$, in which x is water depth and a, b and c constants. The rating curve is expressed by the dark grey solid line in Figure 2.4 and by the linear regression (the black dotted line). Nevertheless, the power function (light gray solid line) and the polynomial regression (grey dashed line) have the best fitting (Figure 2.4). In fact, the RMSE (Root Mean Square Error) estimated is 126.46, 127.47, 124.38 and 121.88, respectively.
For each flood, bedload discharge, Qb, has been calculated in t/day (Figure 2.5). The field observations indicate that bedload was active across the entire cross-section only for big flow discharges, whereas for smaller floods, only the central portion of the river bed was involved. For the first seven floods, the D_{50} varied between 0.296 and 0.626 mm with a mean of 0.43 mm, whereas for the last two floods D_{50} varied from 0.34 to 0.97 mm, with a mean of 0.55 mm. In light of the data collected, it is not definitively possible to establish the functional relationship between liquid and solid discharge. Fig. 5 shows the trend of the monomial function with exponent 1 and 1.24, respectively. The best correlation is expressed by the linear function (black dashed line), with a RMSE equal to 231 (Figure 2.5).
It is worth noticing that the 2017 data, indicated with the black circle in Figure 2.5, correspond to low values of bedload transport despite the flow discharges were relatively high. This result could be accounted for the likely permanent opening of the Fiumi Uniti sluice gate which influenced the sediment movement. With the sluice gate opened there is no chance for upstream sediment accumulation. By contrast, during the 2005-06 measurements, when the sluice gate was closed during low flows, even low discharges were able to remove the sediment accumulated during the closing period of the sluice gate. In order to confirm this hypothesis, the bed shear stress relative to each flow discharge was calculated and it is shown in Figure 2.6.

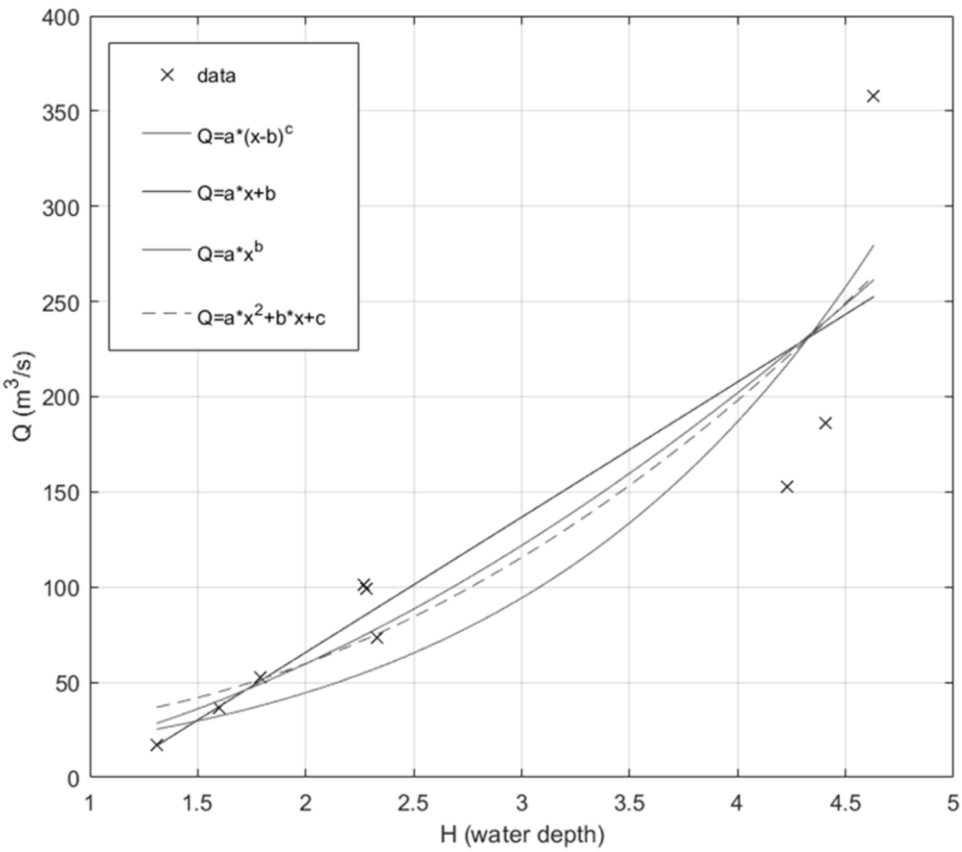


Figure 2.4- Rating curves and summary of measured data. Correlation between water discharge (Q) and water depth (H).

In this case, the linear regression (black solid line, in Figure 2.6) demonstrates the best fitting (RMSE equal to 2.92). Unfortunately, very few studies have investigated the threshold conditions for sandy bed material entrainment, but from our field measurements it is nearly possible to identify the critical flow for the smallest, appreciable bedload transport. This result is compared with the predictions of a few of the most used criteria available in the literature for rivers with sandy or gravelly sand bed material. By applying the classical Shields criterion, as it is represented by Brownlie (1981) and herein reported in Eq. 1, to the threshold flow observed in the field (17.27 m^3/s and tc = 3.73 N/m^2) we have the following values of the critical shear stress: 0.37 N/m^2 (considering D_{50} = 0.43 mm) and 0.47 N/m^2 (considering D_{50} = 0.55 mm), respectively (Table 2.2). The resulting Shields dimensionless parameter, θ_{cr}, resulted 0.050 (both for D_{50} = 0.43 mm and D_{50} = 0.55 mm). These values are lower than the typical θ_{cr} 0.056, values commonly used in bedload transport formulae (Julien, 1995; Hickin, 2010). Along with Shileds-Brownlie criterion for the incipient motion, several other approaches have been considered. These criteria follow an empirical power law of the type $\tau_{cr} = aD^b$, where a and b are two fitting parameters determined by experimental data analysis.

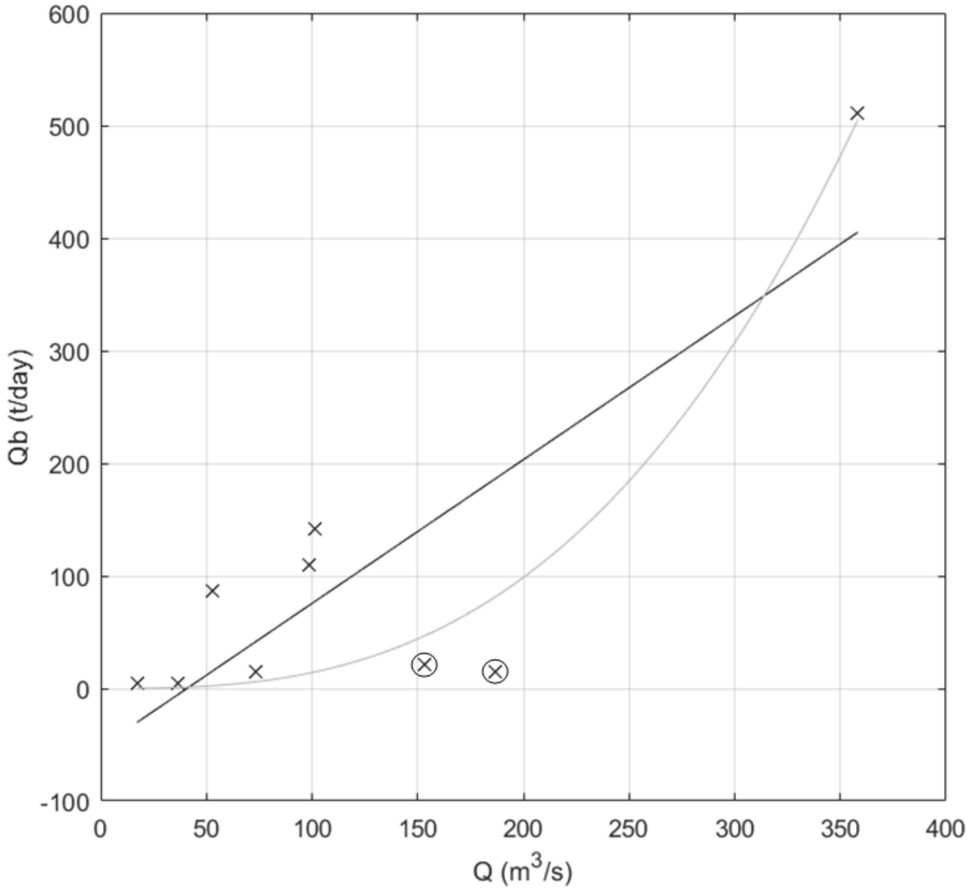


Figure 2.5- Correlation between flow discharge (Q) and sediment discharge (Qb). The circled data refer to the measurements of 2017.

One of the first formulas proposed in the literature are that of Carling (1983) where $\tau_{cr} = 6.33D^{0.38}$ and that of Costa (1983) where $\tau_{cr} = 26.6D^{1.21}$ Hammond et al. (1984) proposed $\tau_{cr} = 55D^{0.42}$. trovare in file word tipi di carattere diversi
Moreover the critical stream power approach, originally represented by Bagnold and then revised by Parker et al. (2011), has been considered as well (Equation 2.6).

$$\boldsymbol{\omega_c = 2861D^{1.5}\log\left(12\frac{H}{D}\right)} \quad \text{[kg/ (ms)]} \tag{2.6}$$

Where:
$\omega_c$represents the threshold value for the stream power,
D is the characteristic particle diameter of the sediment [m];
H is the mean water depth [m].

In this case, the critical shear stress was obtained by the following Equation 2.7:

$$\boldsymbol{\tau_{cr} = g\ (\omega_c/\nu)} \quad [\mathrm{N/\ m^2}] \tag{2.7}$$

in which:
v is the mean flow velocity [m/s],
g is the gravity acceleration, equal to 9:81 m/s^2.

All these criteria for the threshold condition were tested against the field data, considering again both 2005/2006 and 2017 bed material D_{50} (Table 2.2).

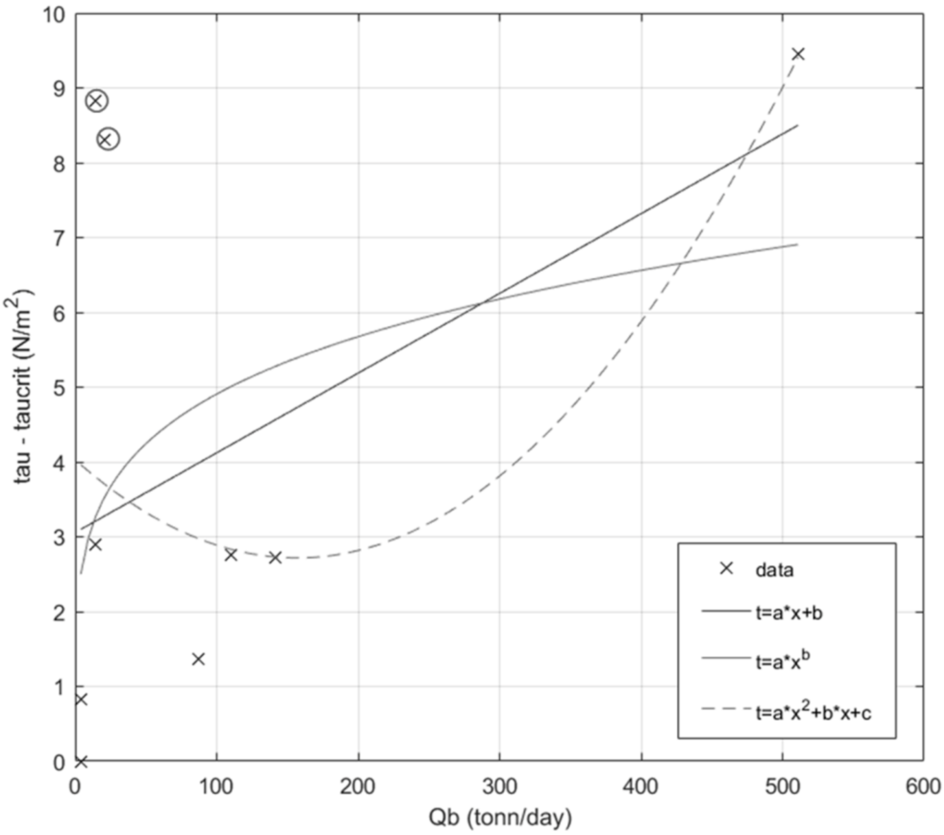


Figure 2.6- Correlation between sediment discharge (Qb) and relative shear stress (τ).The circled data refer to the measurements of 2017.

	$\tau_{cr(D_{50}=0.00043\ m)}$ [N/m^2]	$\tau_{cr(D_{50}=0.00055\ m)}$ [N/m^2]
Shileds-Brownlie (1981)	0.37	0.47
Carling (1983)	3.33	3.65
Costa (1983)	0.06	0.08
Hammond et al. (1984)	1.47	1.63
Bagnold (1980)	3.05	4.31

Table 2.2– Comparison of critical shear stress values obtained by different criteria.

Table 2.2 shows that Carling (1983) and Bagnold (1980) provide the best predictions. In fact their results are very close to the actual value obtained from the field data (3.73 N/m^2). Conversely, the equation of Costa (1983) under predicts critical shear stress and such a large difference can be accounted for by the larger grain size (gravel) of bed material on which it is based. Hammond et al. equation is derived for the finer grain size (5 mm), and though it was based on data from tidal estuaries (Hammond et al., 1984), its results are not satisfactory. The results of Table 2.2 clearly indicated that, with the exception of Bagnold (1980) and Carling (1983), almost all the criteria considered are not reliable to predict critical shear stress in a sand bed costal river like the Fiumi Uniti. These criteria, in fact, are mainly based on gravel and fine gravel-sand mixture and therefore did not take into account the roughness component of bedforms (e.g. dunes) that are present in the study reach. This issue will be matter of future studies. Moreover, the critical shear stress has been calculated also by the Hec-Ras model, using as input flow the threshold discharge observed in the field (i.e., 17.27m^3/s). The energy slope obtained from the model is equal to 0:000135 m=m, therefore the tcr calculated by equation 2.3 resulted equal to 1.73 N/m^2. The value estimated by Hec-Ras is substantially lower than that calculated with field data (3.73 N/m^2).

2.7 Conclusions

The present study, as part of a regional scale project, aims to widen the data set of river sediment supply to the beaches of the whole Emilia Romagna region, which in the last decades were characterized by a marked erosion phenomenon. Bedload measurement campaigns in the Fiumi Uniti River, considered as a representative river of the region, were carried out and are still in progress. The field measurements already performed allowed to obtain significant data related to the sediment size of the bedload, as well as to quantify bedload transport for each flood in association with flow discharge. Finally, the critical shear stress for the sandy bed material entrainment was investigated. The data evidenced a difference in bedload transport between the 2005 and 2006 field campaigns and a new data-set of field measurements undertaken in 2017. In particular, 2017 data demonstrated a decrease in the sediment transport rate, probably due to a difference flow regime at the presence downstream sluice gate. Although the sluice gate was previously maintained opened even during small floods, in recent years it has been kept permanently open. This fact has to be investigated in more detail in order to explain the increase
in bed material median grain size recorded in 2017. A comparison with the results of well-known criteria to predict the threshold conditions for bed particle entrainment indicates that these criteria largely under predict the value of critical shear stress, whereas the classical Bagnold (1980) criterion slightly over predicts the actual threshold. In this context the Carling (1983) seems to be the most acceptable one. Further studies are needed, especially to incorporate the roughness effect of moving dune bedforms.

3. Field data and regional modeling of sediment supply to Emilia-Romagna's river mouths

3.1 Introduction

During the last decades the coast of the Emilia-Romagna region has been affected by a considerable beach retreat phenomena. Given the relevant economic role of the summer tourism, beach protection and reconstruction became crucial for, coastal management (Calabrese and Lorito, 2010; Armaroli et al., 2012). In this region beach erosion is primarily due to the scarcity of sediment supplied by the small local rivers. The importance of small rivers in contributing both to beach stability and marine sedimentation has been previously pointed out for many of the world's coastlines (Milliman and Syvitski, 1992; Inman and Jenkins, 1999). Many factors contributed to the decrease in sediment supply and among them, anthropogenic interventions, such as an increase in deforestation, a change in land use, river bed mining and proliferation of dams, are the most evident (Inman and Jenkins, 1984; Inman, 1985; Kondolf, 1997; Liebault and Piegay, 2001; Grant et al., 2003; Hooke, 2006; Anthony, 2014). Moreover, variations due to climate change such as decreasing of precipitation, runoff and water discharge, directly affected fluvial geomorphology and sediment supply (Billi and Fazzini, 2017). Unfortunately, information about the sediment transport of Emilia-Romagna rivers is limited and restricted to a small number of them (Billi and Salemi, 2004; Ciavola et al., 2005; Ciavola et al., 2010; Billi et al., 2017). In order to enlarge the existing sediment supply dataset, bedload measurement campaigns in representative rivers have been carried out by the authors. Complementary hydrological investigations and hydraulic modeling are ongoing, aiming to define the sediment supply to the whole Emilia-Romagna coast. Since the prediction of the threshold conditions for sediment transport is crucial in modeling the river sediment yield, this paper focuses on this aspect with preliminary results of the undergoing investigations. For this purpose, a wide review of existing bedload field data has been carried out, including new data measured in a recent campaign.

3.2 Study area

The Fiumi Uniti and the Savio are two small river systems located in the southern part of the Emilia-Romagna region (Italy). The Fiumi Uniti, resulting from the unification of the Montone and Ronco rivers, drains the northern Apennines and has a catchment area of about 1000 km^2 (Figure 3.1). The river crosses the city of Cesena and enters the Adriatic Sea between Lido Adriano and Lido di Dante, south of Ravenna (Figure 3.1). The Savio river (catchment area 647 km^2), flows from the Apennines outflowing into the Adriatic Sea between Lido di Classe e Lido di Savio, close to Savio village, south of the former river (Figure 3.1).

The upper catchment of both rivers is underlained by Miocene turbidities consisting of sandstones and marlstones alternation (Amorosi et al., 2002). Alluvial plain deposits consist mainly of Pliocene marine deposits and Quaternary Po river deposits (Amorosi et al., 2002).The climate is typically Mediterranean (Mennella, 1972). Summer is typically dry and precipitation peaks are visible in March and October-November. The annual precipitation rate is 1025 mm and 961.43 mm for the Fiumi Uniti and the Savio, respectively. The mean temperature is practically the same, 13.5 and 13.6 °C.

In the downstream reach of both rivers hydraulic infrastructures such as sluice gate dams are present. Along the Fiumi Uniti river, the Rasponi sluice gate, located 3.5 km upstream

the river outlet, is used to retain water and to prevent upstream salt water migration (Figure 3.1). The Montone and Ronco river tributaries have two dams (San Marco and San Bartolo), located respectively at around 2.8 km and 4 km upstream the confluence (Figure 3.1). Similarly, the Castiglione dam on the Savio river is located 12 km upstream of the river mouth (Figure 3.1). Since all these dams are mainly used for agricultural purposes, the local Land Reclamation Authority (Consorzio di bonifica) keeps them completely opened during the flooding season (i.e. from October to March) and closed, during the dry period (from April to September).
The sediment transport monitoring sites are located in the terminal reaches of both rivers, where they have a sandy bed. The monitoring station of the Fiumi Uniti river is located in Ravenna (Figure 3.2) in correspondence of a suspended pedestrian bridge, almost 8 km upstream of the river outlet. The Savio river's station is located on a road bridge, 3.5 km upstream from the outlet (Figure 3.2).

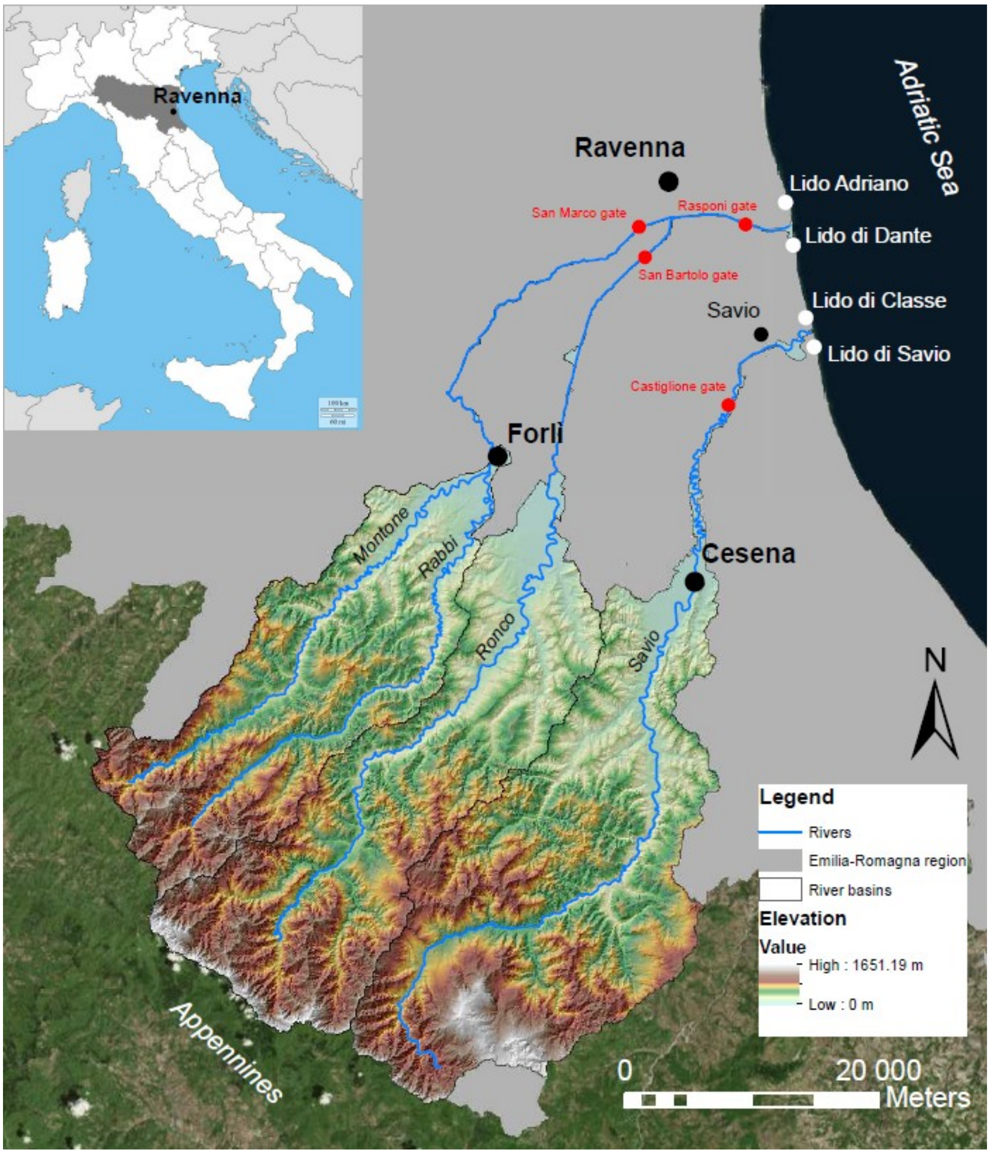


Figure 3.1 - Study area with location of the main artificial structures along the rivers.

Both rivers exhibit a straight channel with rectangular cross-section in proximity of the monitoring stations. The maximum channel width is about 60 m in the Fiumi Uniti and 30 m in the Savio case. The stream bed gradient is 0.00029 m/m in the Fiumi Uniti river (Billi et al., 2017) and 0.0003 m/m in the Savio. Bed material is principally sandy: D_{50} is around 0.55 mm in the Fiumi Uniti and around 0.50 mm in the Savio (i.e. medium to coarse sand).

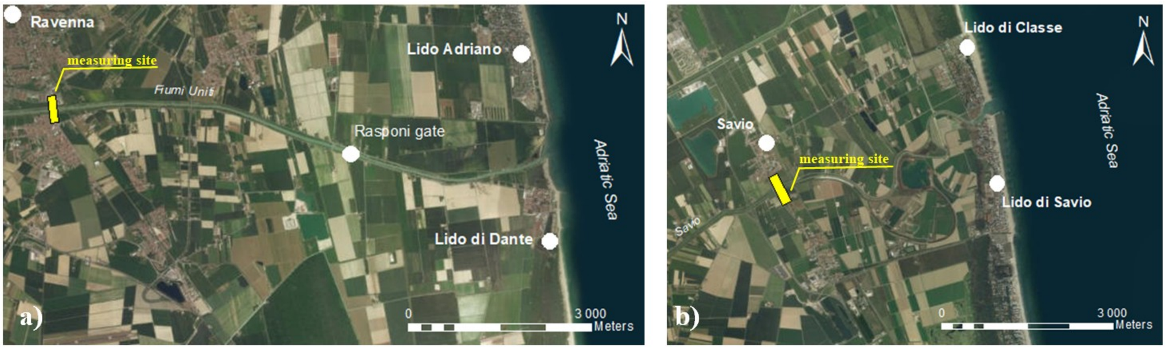


Figure 3.2 - Measuring sites: a) Fiumi Uniti ; b) Savio.

3.3 Methodology

The main research activity essentially included field bedload transport measurement and hydrodynamic modeling. Some of the bedload measurements were carried out in the years 2005-06 and other followed in 2017 and are still in progress. During floods, hydraulic and sediment transport data were collected at fixed verticals, equally spaced along the active cross section (i.e., the portion of the streambed which is actually contributing to bedload transport). Five and three verticals across the river cross-section were established for the Fiumi Uniti and Savio rivers, respectively. A standard USGS AA type current meter measured flow depth and flow velocity. A standard Helley-Smith bedload sampler (US BL-84) with a 76x76 mm intake and 0.1 mm of bag mesh was used for bedload transport sampling. A USGS A type wheel crane helped lowering all the instruments from the bridges. A staff gauge helped to visualize the water level changes. Bedload sampling time varied from 5 to 20 minutes per vertical. Each bedload sample, cleared from vegetation debris and exotic materials, was wet sieved to remove the incidentally present fraction finer than 63 µm (which is considered as a wash-load contribution). The coarse fraction was then dry-sieved for 20 minutes with a standard Ro-Tap shaker and sieves arranged on a ½ phi scale. Complementarily, hydrodynamic modeling was carried out to take into account any backwater effects due to the sluice gate dams (though they were kept constantly open) and the tidal effects (though almost negligible in the areas as the mean tidal range is 0.7 m), and to simulate the hydraulic conditions of the monitored floods. Since the measuring sites show a prismatic channel and experience a slow flow rate changes during floods, a gradually varied flow conditions was assumed for the hydrodynamic simulation, resulting in a shear stress express as Equation 3.1:

$$\tau=\rho g H S \qquad (3.1)$$

Where:

ρ is the density of water;
g is the gravity acceleration;
H is the mean flow depth;
S is the hydraulic gradient.

In particular, the threshold conditions of bedload transport have been examined considering three different approaches. The first refers to Shields's (1936) incipient motion criterion, accounting for Brownlie (1981) and Simoes (2014) reinterpretations of the original Shield's diagram. In fact, as individual grain movement is function of sediment distribution as well as protrusion, packing and grading, they seems to be crucial in sediment transport initiation analysis (Carling, 1983; Hammond et al., 1984; Lorang and Hauer, 2003).Thus a second field-empirical based approach was taken into consideration, and in particular Carling (1983) and Hammond et al. (1984) criteria. To notice that the first refers to gravel bed rivers while the second one refers to tidal channels. Despite these approaches result from different contest and hydrodynamic condition, compared to the current one, both of them evidence deviation from the Shields' curve. These criteria consider an empirical spurious power law function involving stress and representative sediment diameter, as expressed by Equation 3.2:

$$\tau_{cr}=aD^{b} \tag{3.2}$$

where *a* and *b* are calibration coefficients (Lorang and Hauer, 2003) and D is the characteristic particle diameter of the sediment.

The latter is the critical stream power approach (ωc), originally proposed by Bagnold (1981) and revised by Parker et al. (2011), in fact, although bed shear stress is widely used, the unit stream power is more strictly associated with sediment transport (Parker et al., 2011). Bagnold's equation is the following one (Equation 3.3):

$$\omega_{c}=2860.5D^{1.5}\log(12H/D) \tag{3.3}$$

where:
D is the characteristic particle diameter of the sediment;
H is the water depth.

3.4 Results

Twelve floods were monitored at the Fiumi Uniti station and four on the Savio,. During all these floods the sluice gate dams were fully open. Only the floods with an almost unappreciable bedload transport are considered in this study (i.e. comparable to a possible threshold condition of sediment motion). In the Fiumi Uniti river, the weakest flood occurred on 11th of November 2005 with a flow discharge of 17.27 m3 s-1 and a measured bedload lower than 0.06 N m-1s-1. During this flood, the measured mean flow velocity was 0.374 ms-1, with a water depth of 1.31 m.. In the Savio, the weakest flood occurred on 14th of November 2017. Flow discharge was of 19.07 m3 s-1, corresponding to a bedload lower than 0.00016 N m-1s-1, a mean flow velocity of 0.74 ms-1 and a water depth of 1.81 m. This resulted in shear stresses of 1.73 Nm-2 and 3.60 Nm-2 for the Fiumi Uniti and the Savio, respectively. Threshold conditions of bedload transport were calculated according to the three different approaches herein considered (see paragraph 3.3). Referring to the Simoes criterion (2014) a drag coefficient equal to 2.4 was assumed, referring to a mean grain size of 0.5 mm. Concerning the second approach, Carling and Hammond equations were tested using D_{90} as a reference diameter. The use of D_{90} for Carling and Hammond criteria is in respect of their analysis, since in their studies only bigger particles were analyzed (Carling, 1983; Hammond et al., 1984). In case of the third

approach, based on the unit stream power, D_{50} was used calculating the threshold unit stream power value. Table 3.1 reports all these values along with our field results, also in terms of Shields mobility parameter (θcr).
Figure 3.3 reports different criteria on θ_{cr} –vs- Re* log-log plotting chart; in particular the actual field observation are plotted accounting for both D_{50} and D_{90} (see Table 3.2)

Table 3.1 - Comparison of threshold sediment transport condition and field data.

	$\tau_{cr\ FIUMI\ UNITI}$ [N/m^2]	θ_{cr} [-]	$\tau_{cr\ SAVIO}$ [N/m^2]	θ_{cr} [-]
Field data	1.73	0.22	3.60	0.42
Shields-Brownlie	0.47	0.05	0.42	0.05
Shields-Simoes	1.38	0.10	1.41	0.11
Carling	4.55	0.29	4.36	0.31
Hammond et al.	2.07	0.13	1.98	0.14
Bagnold-Parker	4.31	0.10	1.95	0.46

Table 3.2- Shields' parameters calculated with D_{50} and with D_{90} referred to both rivers.

	D_{50}		D_{90}	
	Re*	θ_{cr}	Re*	θ_{cr}
Fiumi Uniti	21.83	0.20	40.69	0.11
Savio	29.78	0.45	52.65	0.25

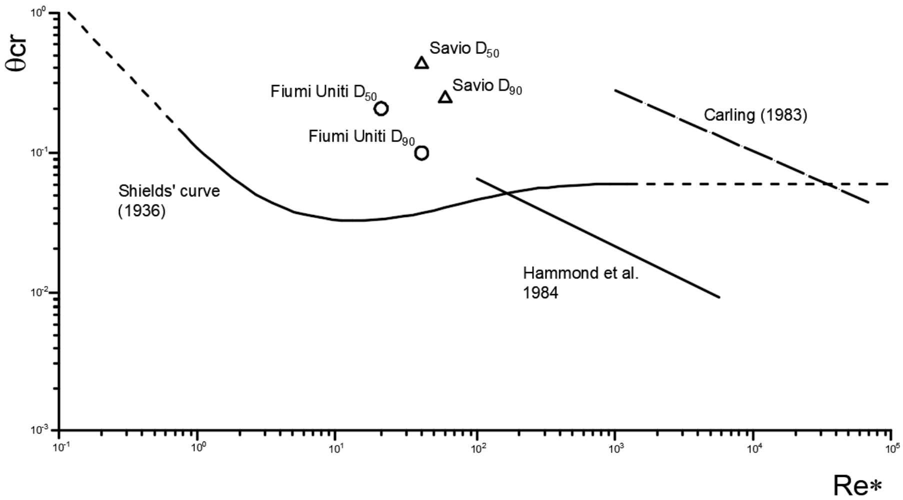


Figure 3.3 - Fiumi Uniti and Savio river field data compared with Shields [19] and Carling, Hammond et al. [23] criteria.

3.5 Discussion

The sediment transport condition of sediment motion analyzed and reported in Table 3.1 puts in evidence a significant inaccurate estimation of critical shear stress values, compared to the field data. Moreover considering the results related to the Shields dimensionless parameter θcr, all criteria show a big difference in values (i.e. across an order of magnitude), except for Bagnold (modified by Parker) and Carling. Shields and revised Shields criteria (Brownlie and Simoes) evidence their non-applicability, especially explained by the low value of θcr respect the ones obtained from field data. Shields approach is mainly based on single grained sediment and does not consider irregularly shaped grains. Instead the criterion of Carling takes into account factors like grain size, grading, packing and protrusion which are completely excluded in Shields and revised Shields criteria. Moreover it is referred to narrow (aspect ratio < 11) gravel (or even coarser) bed steep channels, and it may leads to the reported discrepancies with our field data. Also Hammond et al. criterion consider protrusion and packing factors but their model was implemented on small tidal estuaries with a bi-modal, gravel-prevailing, bed. As far as Bagnold criterion is concerned, Parker already puts noticed that the unit stream power is more variable with slope than critical mean shear stress. And even if the shear stress values calculated with Bagnold criterion still differs from our field data, the critical Shields parameter value (θcr) seems reasonable.

3.6 Conclusions

The aim of this research, as part of a regional scale project, is to widen the dataset of river sediment supply to the Emilia-Romagna beaches, which are affected by severe erosion. Bedload measurement campaigns carried out in the Fiumi Uniti and Savio rivers allowed to estimate the critical shear stress for bed material incipient motion in sandy-bed rivers. The definition incipient motion condition obtained from field data was compared with well-known criteria available in literature, such as the classical Shields approach as well as alternative methods. The results identified a noticeable difference between field, theoretically computed critical values.
Results point out that the tested criteria have the tendency to largely under predict the value of critical shear stress, whereas others slightly over predicts the actual threshold. Furthermore, considering the results related to the Shields dimensionless parameter θ_{cr}, all criteria provide big discrepancies (i.e. across an order of magnitude), except for Carling criterion which is strongly preferred. One of the main principles which are accounted by Carling approach it the fact that it does consider irregularly shaped grains. In fact it examines factors such as grain size, grading, packing and protrusion which are completely excluded in alternative methods. Moreover Carling approach has been developed in narrow (aspect ratio, i.e. width to depth ratio < 11) gravel (or even coarser) bed steep channels. Despite this fact, it seems to be the most suitable approach for the present study cases. Except for grain sediment size, the fact that it refers to narrow streams could be taken into consideration as element of importance for further studies. On this purpose further investigation are necessary, possibly including the bed roughness effect due to the presence of moving bedforms.

4. Dune contribution to flow resistance in alluvial rivers

4.1 Preamble

In nature, sand bed streams rarely present flat beds. In fact, streambed local deformations develop as symptoms of sediment transport, resulting in sequences of erosion and deposition. Observations both in natural rivers and flume have confirmed a typical sequence of bedforms develops with the increase of flow velocity.. The evolution of bedforms occurs under a set of flow regimes described by the Froude number F, expressed as in Equation 4.1:

$$F = \frac{U}{(gD)^{\wedge}0.5} \tag{4.1}$$

where: U is mean flow velocity [m/s]; g is gravity acceleration [m/s^2] and D is the water depth [m].

Based on these stages of flow, the most recognized sequence are ripples, dunes, plane bed and antidunes, but one of the most common bedform observed in natural sand bed rivers are dunes, which form under lower flow regime conditions($F < 1$). Dunes are bedform out of phase with the flow surface and form in a range of sediment sizes sand to to gravel (Dinehart, 1992; Seminara, 1995; Best, 1996; Carling, 1999; Kleinhans, 2001, 2002; Carling et al., 2005). In general dunes are characterized by values of length in a range between 1 and 16 times the flow depth, while their height stands between 1/40 and 1/6 of flow depth (Allen, 1982). Field observations permitted to distinguish symmetric and asymmetric dunes (Venditti, 2013). Large scale symmetric dunes have been observed mainly in large rivers and estuaries (Kostaschuk and Villard, 1996; Roden, 1998; Carling et al., 2000), whereas asymmetric dunes are more common in smaller channels and laboratory flumes (Kostaschuk and Villard, 1996). While symmetric dunes are characterized by a sinusoidal geometry, asymmetric ones are characterized by a gentle stoss-side and a steep lee-sides, due to the interaction between unidirectional flow and the sediment transport (Leliavsky 1955).

Since dunes are an important roughness element in providing resistance to flow; therefore, the knowledge of their geometry is fundamental in predicting flow resistance, sediment transport, and deposition. Dune dynamic has been widely investigated in the laboratory, whereas field studies and data are still very scanty (Best, 2005).
One of the most studied topic is the role of bedforms, and dunes in particular, in flow resistance. As this is considered a key component, significant effort has been invested in the study of river dunes and the resulting roughness development during floods, predominantly in laboratory flume. Total resistance is commonly considered to be provided mainly by grain and form resistance (Einstein and Barbarossa, 1952; Vanoni and Brooks, 1957; Smith and McLean, 1977).,Several methods were proposed for separating grain and form resistance e.g. Einstein, 1950; Einstein and Barbarossa, 1952; Engelund and Hansen, 1967; van Rijn, 1984; Nelson and Smith, 1989; Garcia and Parker, 1993; Wright and Parker, 2004a, b. The results of these studies indicate that total flow resistance is up to 2-3 times grain resistance (Venditti, 2005). As evidenced by Venditti (2005), the main difference between these methods is that one type of approach uses empirical relation, while the other uses bedform properties to calculate shear stress components. Einstein (1950), Einstein and Barbarossa (1952), Engelund and Hansen

(1967), Engelund and Fredsoe (1982), Wright and Parker (2004a) are some of the most known methods which use the first approach; while Fredsoe (1982), van Rijn (1984), Nelson and Smith (1989), Garcia and Parker (1993) follow the second one.
In this PhD work, a deepening related to this topic has been done, implementing a new approach to estimate the effect of dunes on flow resistance (Chapter 6). The resulting equation of the model presented, which takes into account flow pattern and bed form geometry, has been validated using a large selection of field data. Further, in Chapter 7, a comparison of the proposed approach with two of the most well known criteria (Engelund and Hansen, 1967; Hengelund and Fredsoe, 1982) has been developed on the two river systems, object of the research.

4.2 Introduction

In alluvial channels, flow resistance is the result of several factors, including the presence of bends, vegetation, local acceleration due to flow unsteadiness, and channel geometry variations, in addition to bed surface roughness and bed forms drag. Morphological evolution of active channels also affects flow resistance, since it is responsible for the evolution of large-scale bed forms (i.e., dunes and bars). Since the 1950s great efforts have been devoted to river morphodynamic models. Most of them consist of coupled systems of depth-averaged flow mass and momentum equations, a bed evolution equation, as well as a sediment transport formula. Valuable state-of-the-art technologies for river sedimentation and morphology modeling may be found in [Cao et al., 2002; Wang and Wu, 2004). These conventional morphodynamic models mainly refer to nonlinear shallow water equations, Exner equation, and an empirical formula for sediment transport. Considerable uncertainty derives from the large number of such formulae available in the literature. Moreover, sediment transport in open channel flows includes bedload and suspended load; but, although different mechanisms govern these two modes of transport, a reliable method to account for both of them has not yet been provided (Amoudry and Souza, 2011). In conventional models further uncertainty arises from the lack of understanding of some fundamental mechanics related to sediment transport. Among them, the effect of bed slope, which has been considered by Maldonado and Borthwick, who recently proposed a simplified two-layer model (Maldonado and Borthwick, 2018) bedload-dominated scenarios. Alternatively, morphodynamic models based on two-phase and two-layer approaches may also be considered (Abril et al., 2012;Li et al., 2013; Iverson and Ouyyang, 2015) Although they are scientifically more insightful than the conventional approaches, they prove to be mathematically more complex and computationally more demanding.
Even in the case of steady and quasi-uniform flow, confined within the main active channel and involving non-cohesive sediment, most of the existing resistance formulae (e.g., Einstein and Barbarossa,1952; Engelund and Hansen, 1967; Van Rijn, 1984) commonly provide inaccurate predictions of flow resistance. This leads to considerable error in stage-discharge prediction, both in terms of water depth (±20%), flow velocity (±15%) (Karim, 1990) or bed friction, for which the difference between measured and predicted values may be ±50% (White et L., 1980) and, where field data is used, discrepancy is even wider (Yang, 2008; Billi et al., 2017)). Since the middle of the twentieth century this topic has attracted the interest of many scientists and several criteria-based methods have been proposed, based on linear and non-linear approaches.
In the latter case (e.g., Yalin, 1977; Van Rijn, 1982;Karim, 1990; Yu and Lim, 2003) the resistance coefficient is kept as a single factor and the method does not require any

knowledge of bed form geometry (i.e., empirically-based approach). By contrast, the linear approach originally introduced by Meyer-Peter and Mueller (1948) considers an overlapping of effects. In fact, flow resistance depends on forces acting on individual particles (i.e., skin friction) and on forces affected by bed form configuration (i.e., form drag). Among the others Azareh et al. (2014) investigated the contribution of form friction to the total friction factor, providing evidence that form friction contributes up to 65% of the total friction factor of a gravel-bed river.

Since the bed form contribution to flow resistance mainly depends on dune geometry and flow conditions, recently big efforts have been devoted to researching bed form geometry. Yalin and Karahan (1979) found that the spacing of dunes was dependent on the relative depth (i.e., d_{50}/y with d_{50} being bed material average grain size and y water depth). Julien and Klaassen (1995) made a field investigation on the Meuse River and the Rhine River during large floods, and discovered that dune steepness remains relatively constant with discharge, and suggested a linear proportion between wavelength and water depth. The proposed linear coefficient differs from the one empirically proposed by Yalin (1964b), which either takes into account flume and river data, or theoretically derived (Yalin, 1977). Agarwal et al. (2001) stressed that, for specific range of relative depth, dune spacing may greatly differ from what has been suggested by Yalin (1964a;1977), or by Julien and Klaassen (1995). Aberle et al. (2010) used a statistical approach to investigate bed forms during different flood conditions in the Elbe River showing how statistical parameters may be used to predict the flow-dependent bed roughness.

Despite the uncertainties regarding the correct representation of dune geometry, the linear approach to flow resistance in presence of bed forms still remains fascinating, in particular because it offers the possibility to explore each component (i.e., skin roughness and bed form drag) separating the independent parameters involved in the process. Recently, Yang (2005) proposed an empirical approach identifying the effective toss length over the dune where the flow is in contact with the bed (i.e., associated with the skin roughness), and the separation zone behind the dune crest, which is responsible for the bed form contribution to flow resistance. An analytical model was introduced by Engelund (1966) and Yalin (1964b). These authors considered the form drag as a result of a sudden flow expansion on the bed form lee side. More recently, Ferreira Da Silva and Yalin (2017) generalized two modes of bed forms drag (by virtue of additivity of losses), also taking into account the ripples superimposed on dunes. In any case, they applied momentum, energy and mass balance equations to a reference bed form, assuming that the effects of a bed form on the flow are comparable to a sudden expansion of a pipe flow.

In this paper, a semi-analytical model for the bed form drag is proposed, considering the effects of a sudden expansion of a free surface flow rather than of a pressure flow. The energy balance equation is applied to a 2D steady flow over a reference dune bed. In order to account for the actual flow and bed pattern, an empirical coefficient is introduced in the bed form drag formula and its dependence on dune geometry is analyzed. The skin resistance is calculated assuming Pandtl–Karman velocity distribution and, according to superposition of the effect, the hydraulic energy gradient is calculated. The model is validated by comparing observed and calculated energy gradients. Since this paper focuses on sand rivers in presence of dunes, field data related to sand streams and large sand rivers are considered, accounting for a large span of different hydraulic and sedimentary conditions.

4.3 Flow resistance

In alluvial rivers with sediment transport, flow resistance is generated by surface roughness and bed forms, which act as a macroroughness. In presence of macroroughness the average velocity distribution and cross section geometry are not irrelevant. In fact, the velocity distribution and its gradient, as well as bed shear stress, vary along the channel reach, accounting for actual geometry and flow field.

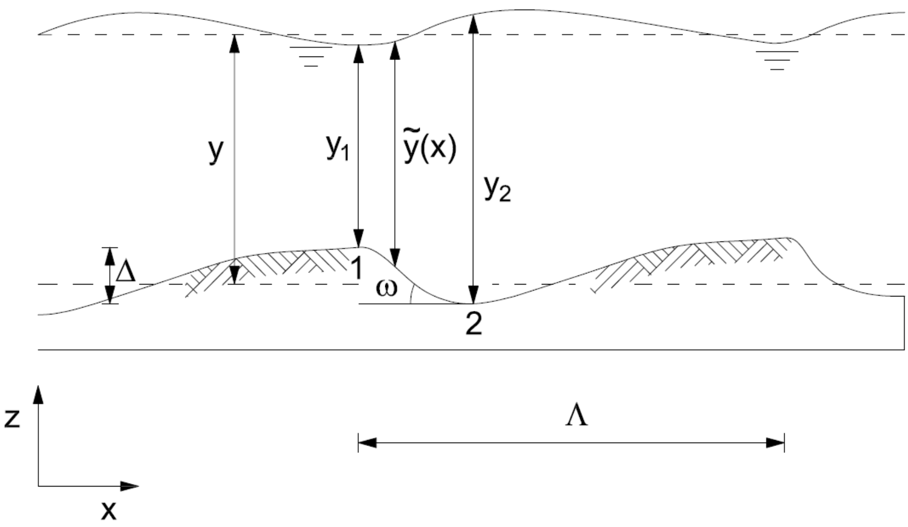


Figure 4.1 - Schematic representation of 2-D flow in presence of dune.

In a two-dimensional two-phase steady flow in presence of dunes (see Figure 4.1), the velocity distribution relates to the following characteristic parameters (Yalin, 1977; Ferreira Da Silva and Yalin , 2017).

$$U, y, S, \nu, \rho, \gamma_s, d_s, \Delta, \Lambda, g \tag{4.2}$$

where U is the mean flow velocity, y the mean flow depth, S the friction slope, ν and ρ the kinematic viscosity and the density of water, respectively, γ_s the specific weight of immersed sediment, d_s the representative sediment diameter, Δ the dune height, Λ the dune length, and g the gravity acceleration. Therefore, it must hold a dimensionless general relationship of dependency on the following seven dimensionless parameters:

$$\varphi\left(S, \frac{y}{d_s}, \frac{U\cdot y}{\nu}, \frac{U^2}{g\cdot y}, \frac{\gamma_s}{\rho\cdot g}, \frac{\Delta}{y}, \frac{\Lambda}{y}\right) = 0 \tag{4.3}$$

Assuming a constant value for the relative immersed weight of sediment in natural channel**s**, and introducing the following dimensionless variables (i.e., relative submergence *Z*, Reynolds' number *Re* and Froude number *F)*:

$$Z = \frac{y}{d_s}; Re = \frac{u\cdot y}{\nu}; F^2 = \frac{U^2}{g\cdot y} \tag{4.4}$$

the following relationship for the global friction slope is obtained:

$$S = \tilde{\varphi}\left(Z, Re, F^2, \frac{\Delta}{y}, \frac{\Lambda}{y}\right) \tag{4.5}$$

So far, no theory analytically derives such a function, even for the simple reference case of steady, uniform 2D flow in a steady sediment transport condition. A possible approach consists in separating the global flow resistance into two individual contributions: surface roughness and macroroughness (i.e., bed form). Meyer-Peter and Muller (1948) and Einstein (1950) proposed a linear superposition assuming that the total bed shear stress (τ) could be separated into two contributions—plane bed stress (τ') and additional shear stress—associated with the bed forms (τ''):

$$\tau = \tau' + \tau'' \tag{4.6}$$

According to the assumed superposition of the effects, τ' is equal to the shear stress acting on a sand-bed plane having the same grain-size distribution and the same hydrodynamic condition (i.e., velocity and flow depth). Later, Engelund (1966) proposed that the total mean energy gradient per unit length of the stream (S) could be considered as a contribution of a friction loss (S') due to the stress acting along the dune surface and of a cumulative loss due to a sudden flow expansion just downstream from the dune crest (S''):

$$S = S' + S'' \tag{4.7}$$

and *Equation (6.5*) leads to:

$$\gamma y S = \gamma y S' + \gamma y S'' \tag{4.8}$$

Therefore, the total head loss (ΔH) over a stream reach of length L results from head loss due to the grain resistance ($\Delta H'$) and head loss related to the dune ($\Delta H''$):

$$\Delta H = S \cdot L = \Delta H' + \Delta H'' = S' \cdot L + S'' \cdot L \tag{4.9}$$

4.4 Grain contribution to flow resistance

The characteristics of the roughness elements located along the wetted perimeter affects the vertical profile of the downstream component and the resistance to flow. For fully developed turbulent flow over a rough sand-bed plane ($\Delta/y = \Lambda/y = 0$), *Re* and *F* become irrelevant and the stream velocity profile can be expressed by a logarithmic law:

$$\frac{u}{u_*'} = \frac{1}{k}\ln\left(30\frac{z}{k_s'}\right) \tag{4.10}$$

where u is the flow velocity at elevation z above the bed, k = 0.4 is the Von Karman's constant, $u_*' = \sqrt{\tau'/\rho}$ is the shear velocity related to the skin roughness and k_s' is the equivalent grain roughness. The vertically-averaged velocity U corresponds to the local velocity u at the relative depth $z/y = e^{-1} = 0.368$. By integration over the flow depth, it results:

$$\frac{U}{u_*'} = C' = \frac{\chi'}{\sqrt{g}} = \frac{1}{k}\ln\left(\frac{11 \cdot y}{k_s'}\right) \tag{4.11}$$

where χ' is the Chezy's coefficient and C' the dimensionless conveyance coefficient related to the skin roughness k_s'. According to different Authors, k_s' is proportional to a characteristic sediment size d_s, with subscript "*s*" being equal to 35, 50, 65, 84, 90 (i.e., the percentage of the finer particle size distribution by weight). Typically, k_s' ranges between 1.25 d_{35} and 5.10 d_{84} (Einstein and Barbarossa, 1952; Engelund and Hansen, 1967; Ackers and White, 1973; Kamphuis, 1974; Hey, 1979; Van Rijn, 1982; Cao and Carling, 2002; Wang and Wu, 2004; Amoudry and Souza, 2011; Abril et al., 2012; Li et al., 2013; Iverson and Ouyang, 2015; Maldonado and Borthwick, 2018), whereas Millar (1999) concluded that in gravel streams there is no significant difference between using d_{35}, d_{50}, d_{65}, d_{84} or d_{90}. In the present analysis, the mean grain size diameter d_{50} is assumed as a characteristic sediment size. Different values for k_s were considered and the most appropriate value resulted $k_s = 2 \cdot d_{50}$.
In terms of slope friction, Equation (6.10) results:

$$S' = \frac{(u_*')^2}{g \cdot y} = \frac{U^2}{C'^2 \cdot gy} = \frac{F^2}{\left[\frac{1}{k}\ln\left(\frac{11 \cdot y}{k_s'}\right)\right]^2} \tag{4.12}$$

which is consistent with Equation (4.5).

4.5 Sand dune contribution to flow resistance

In a two-dimensional two-phase steady flow in presence of dunes (see Figure 4.1), the momentum balance equation is applied to the reference control volume of a portion of 2D dune bed. The control volume is bounded between cross section 1 upstream, in correspondence to the crest of the dune, and cross section 2, downstream, in the trough zone where the streamlines are assumed to be parallel to the bed:

$$\mathbf{P} + \mathbf{G} + \mathbf{M} = \mathbf{0} \tag{4.13}$$

where **P** represents the pressure vector acting on the boundary of the fixed control between the two consecutive cross sections 1 and 2, **G** represents the mass force vector acting on the control volume and **M** represents the momentum flux vector through the boundary. The x-component of the previous equation gives:

$$P_1 - P_2 + P_L \sin(\omega) - T_L \cos(\omega) + G \sin(\sigma) = M_2 - M_1 \tag{4.14}$$

being P_1, P_2, P_L the pressure force acting on the upstream cross section, downstream cross section and on the lee side of the dune, respectively. T_L is the shear stress on the lee side of the dune, M_1 and M_2 the momentum flux across the upstream and downstream cross sections 1 and 2, respectively, ω the angle between the lee dune side and the horizontal, and σ the average bed slope.
In a sand-bed river channel, the active longitudinal component of mass force $G \cdot \sin(\sigma)$ can usually be disregarded with respect to the other forces. Since streamlines are parallel to the bed at cross sections 1 and 2, and the velocity in the flow separation zone may be

considered null, hydrostatic pressure holds at both cross sections 1 and 2. The surface force component related to the shear stress (T_L cos(ω)) is already included in the grain roughness component of the total resistance to flow. Introducing the momentum coefficient β (accounting for non-uniform velocity distribution over the cross section), the momentum balance equation per unit width of the channel becomes:

$$\frac{1}{2}\gamma y_1^2 - \frac{1}{2}\gamma y_2^2 + P_L \sin\omega = \rho\beta_2 U_2^2 y_2 - \rho\beta_1 U_1^2 y_1 \tag{4.15}$$

where y_i is water depth at cross section (i = 1,2). The energy balance equation applied between cross sections 1 and 2 results:

$$\Delta H'' = H_1'' - H_2'' = z_1 - z_2 + y_1 - y_2 + \frac{1}{2g}\alpha_1 U_1^2 - \frac{1}{2g}\alpha_2 U_2^2 \tag{4.16}$$

where z_i (i = 1,2) is the bed level and α_i the Coriolis coefficient (i = 1,2) referred to cross sections 1 and 2. For the sake of simplicity from now on $\alpha_1 = \alpha_2 = \beta_1 = \beta_2 = 1$. The location of the stagnation point is not known a-priori and, taking into account for a mild slope of the toss face of the dune, it may be assumed $z_1 - \Delta = z_2$. Moreover, considering a reference dune bed pattern having a vertical dune lee side (i.e., ω = 90°), Equation (15) becomes:

$$\Delta H'' = y_1 + \Delta - y_2 + \frac{U_1^2}{2g} - \frac{U_2^2}{2g} \tag{4.17}$$

and momentum balance Equation (14) leads to:

$$\frac{1}{2}\gamma\left(y_1 + \Delta\right)^2 - \frac{1}{2}\gamma y_2^2 = \rho U_2^2 y_2 - \rho U_1^2 y_1 \tag{4.18}$$

Disregarding the difference in water levels between cross sections 1 and 2, Equation (4.17) results:

$$\Delta H'' = \frac{1}{2g}\left(U_1^2 - U_2^2\right) \tag{4.19}$$

It is convenient to refer to the mean water depth:

$$y = \frac{1}{\Lambda}\int_\Lambda \tilde{y}(x)\cdot dx \tag{4.20}$$

where $\tilde{y}(x)$ is the local water depth (see Figure 4.1). For a dune bed-dominated river, the following approximation is considered (Yalin, 1977):

$$y = y_1 + \frac{\Delta}{2} = y_2 - \frac{\Delta}{2} \tag{4.21}$$

Introducing the water discharge per unit width of the channel:

$$q = U_1 y_1 = U_2 y_2 \tag{4.22}$$

and accounting for the mean water depth *y (*Equation (4.21)), results:

$$\Delta H'' = \frac{q^2}{2g}\left(\frac{1}{\left(y-\Delta/2\right)^2}-\frac{1}{\left(y+\Delta/2\right)^2}\right) \tag{4.23}$$

After algebraic simplification:

$$\Delta H'' = \frac{q^2}{gy^2}\Gamma_{\Delta/y} \tag{4.24}$$

where the dune geometric correction function $\Gamma_{\Delta/y}$ is:

$$\Gamma_{\Delta/y} = \frac{2\left(\Delta/2y\right)}{\left[1-\left(\Delta/2y\right)^2\right]^2} \tag{4.25}$$

It is worth noting that Yalin (1964) and Engelund (1966), independently, applied a 1-D momentum conservation equation to the same dune bed reference pattern, but they assumed a constant pressure over the cross sections 1 and 2 (i.e., Borda–Carnot's theorem for pressure pipe flow) rather than a hydrostatic pressure distribution, and they obtained:

$$\Delta H^{P}{}'' \approx \frac{q^2}{gy^2}\cdot 2\left(\Delta/2y\right)^2 \tag{4.26}$$

where $\Delta H^{P}{}''$ indicates the bed form energy loss according to the pressure pipe flow approach. In this case:

$$\Gamma^{P}_{\Delta/y} = 2\left(\Delta/2y\right)^2 \tag{4.27}$$

A comparison of Equations (4.24) and (4.26) demonstrates the difference between the approach based on free surface flow instead of pipe flow, when the effects of sudden flow expansion in presence of dune is considered (Figure 4.2). Over the typical range of relative dune depth Δ/y = 0.1 − 0.3 the ratio between the two approaches, in terms of energy loss, varies from around 10 to 6. It supports the use of Equation (4.24) instead of Equation (4.26).

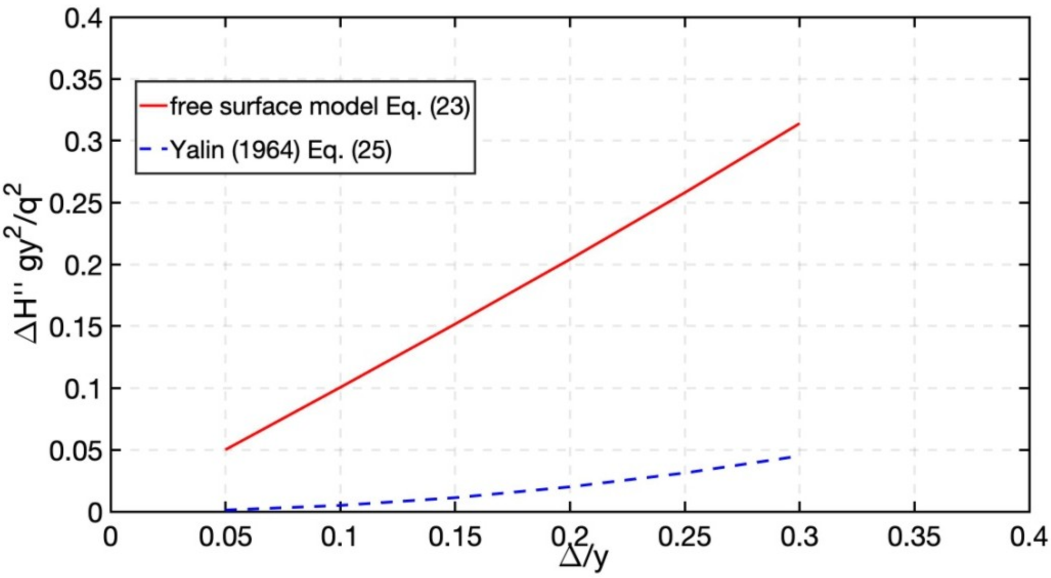


Figure 4.2 - Difference between free surface flow approach, and pressure pipe flow approach.

Using $\Delta H'' = S''\cdot\Lambda$, and after multiplying and dividing by the mean water depth (y), the energy slope related to the dune form drag results:

$$S''=\frac{\Delta H''}{\Lambda}=F^2\frac{y}{\Lambda}\Gamma_{\Delta/y} \tag{4.28}$$

Empirical coefficient for bed form drag

Tokyay and Altan-Sakarya (2011) tested the local energy losses at negative steps in subcritical open channel flows and demonstrated that energy losses are higher for inclined steps than for abrupt vertical steps. Van der Mark (1978), following experimental results of flow over a single bed form with a vertical lee face concluded that an empirical coefficient should be applied to Equation (6.25) in order to take into account that flow velocity distribution is not uniform over the cross-section at the cross-sections 1 and 2. Engel (1981) made experiments on the length of flow separation over artificial dunes in a flume, using fixed bed geometries. He showed that the separation length is independent of the Froude number, but it is dependent on the relative dune height Δ/y. Shen et al. (1990) carried out laboratory experiments on rigid bed forms using both a rough and smooth surface. He showed that pressure drag coefficient relative to the bed form depends on the relative dune height, and on the dune slope steepness, whereas flow velocity and grain roughness does not influence it.

Therefore, the empirical correction coefficient κ, which is a function of the dune steepness $\delta = \Delta/\Lambda$, is introduced into Equation (4.28*),* to also take into account any differences between the theoretical and the actual flow field and bed form geometry, as a consequence of the assumptions so far introduced:

$$S''=\kappa(\delta)\cdot\frac{\Delta H''}{\Lambda}=\kappa(\delta)\cdot F^2\frac{y}{\Lambda}\cdot\Gamma_{\Delta/y} \tag{4.29}$$

Using the breakdown of the energy slope S into its two components related to skin roughness S' and bed form drag S'' *(*Equation (4.7)), Equation (4.29) gives the following expression for the empirical coefficient κ:

$$\kappa(\delta) = \frac{(S - S')}{F^2 \frac{y}{\Lambda} \Gamma_{\Delta/y}} \tag{4.30}$$

Once the bed dune geometry is known, considering the measured energy slope *S,* and after *S*' is calculated by Equation (4.12) (assuming $k_s = 2 \cdot d_{50}$), it is possible to determine the empirical coefficient κ. A selection of 132 field data collected on 7 different sand-bed rivers with dune bed forms is considered (see Table 4.1). It refers to the Savio and Fiumi Uniti rivers (Cilli et al., submitted), Calamus River (Gabel, 1193), Missouri (data from Shen 1978, as reported by Brownlie (1981)), Jamuna and Parana (Julien 1992), Bergsche Maas (data from Adriaanse 1986, as reported by Julien (1992)), and Meuse (Julien 1992). The database reports for each data set the number of measurements *N*, and the range of the following observed parameters: water discharge *Q*, mean depth *y*, mean flow velocity *U*, measured energy gradient *S,* Froude number *F*, mean grain size diameter d_{50}, dune height *Δ* and dune length *Λ.* Despite the relative scatter of field data, Figure 4.3 shows how the drag coefficient decreases with increased dune steepness. Some outliers are present, corresponding to few data recorded on the Jamuna and Missouri rivers (Julien 1992),. characterized by very low values of dune steepness (i.e., $\Delta/\Lambda < 0.01$), and filtered in the fitting procedures. The best fitting equation is:

$$\kappa(\delta) = m \cdot (\delta)^n \tag{4.31}$$

with $m = 0.053$ and $n = -0.2$.

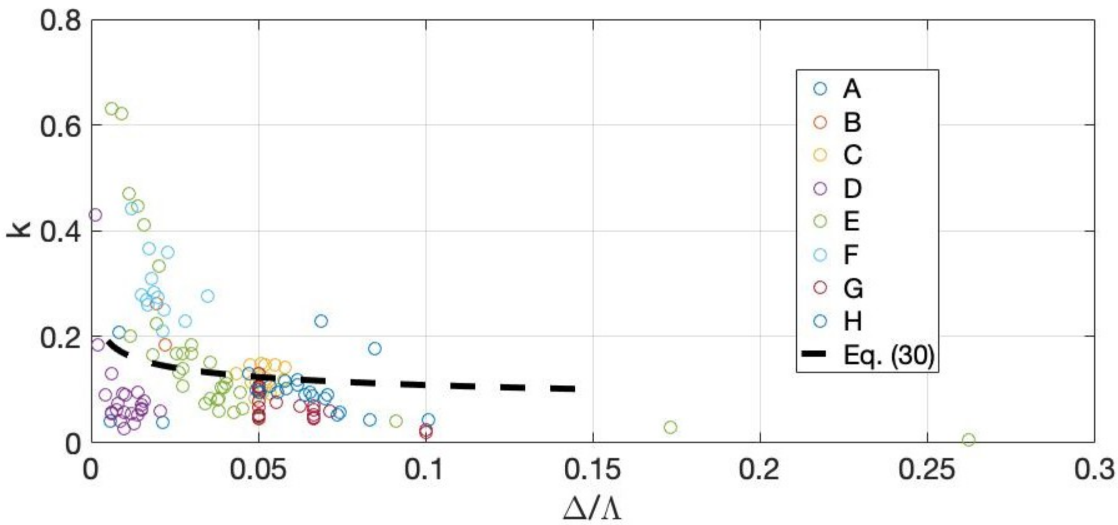


Figure 4.3 - Empirical coefficient κ as a function of dune steepness δ = Δ/Λ. (Data A–H see Table 4.1).

Table 4.1 - Summary of field data used to determine empirical coefficient κ (Equation (29)). Each column reports maximum and minimum value.

Code	River	N	*Q* (m^3/s)	*Y* (m)	*U* (m/s)	*S* (m/km)	*F* (-)	d_{50} (mm)	d_{90} (mm)	*Δ* (m)	*Λ* (m)
A	Fiumi Uniti	22	358.40-21.17	4.72-1.31	1.66-0.20	0.139-0.002	0.24-0.04	0.655-0.390	2.100-0.630	0.28-0.10	17.53-13.1
B	Savio	9	132.06-7.04	3.58-1.77	1.50-0.21	0.354-0.012	0.28-0.05	0.548-0.412	1.702-0.694	0.16-0.12	7.16-6.14
C	Calamus	18	1.73-0.82	0.61-0.34	0.77-0.61	1.100-0.680	0.34-0.29	0.410-0.310	-	0.20-0.10	4.05-2.02
D	Missouri	25	1817.20-179.20	4.99-2.77	1.76-1.28	0.185-0.125	0.32-0.22	0.266-0.190	0.311-0.217	2.07-0.58	735.18-57.91
E	Jamuna	33	10000-5000	19.50-8.20	1.50-1.30	0.070	0.17-0.09	0.200	-	5.10-0.80	251.00-8.00
F	Parana	13	25000	26.00-22.00	1.50-1.00	0.050	0.10-0.07	0.370	-	7.50-3.00	450.00-100.00
G	Zaire	29	28490-284	17.60-6.80	1.69-0.32	0.345-0.042	0.16-0.03	0.545-0.430	1.900-0.430	1.90-1.20	450.00-90.00
H	Bergsche Maas	20	2160.	10.50-5.80	1.70-1.30	0.125	0.20-0.13	0.520-0.210	-	2.50-0.40	50.00-6.00

According to Equation (4.30), the estimated drag coefficient *k* depends on the skin energy slope (*S*') which, in turn, depends on the equivalent roughness k_s. Among the selected field data (river A–G, Table 4.1*),* Fiumi Uniti, Savio, Missouri and Meuse River reported information about sediment gradation or d_{90}. Figure 4 shows a comparison between the drag coefficient *κ* obtained considering equivalent roughness $k_s = 2 \cdot d_{50}$ or $k_s = 3 \cdot d_{90}$.
The general trend remains confirmed, and it may be concluded that coefficient *κ* is almost independent of the equivalent skin roughness adopted to estimate the grain resistance contribution *S'*.

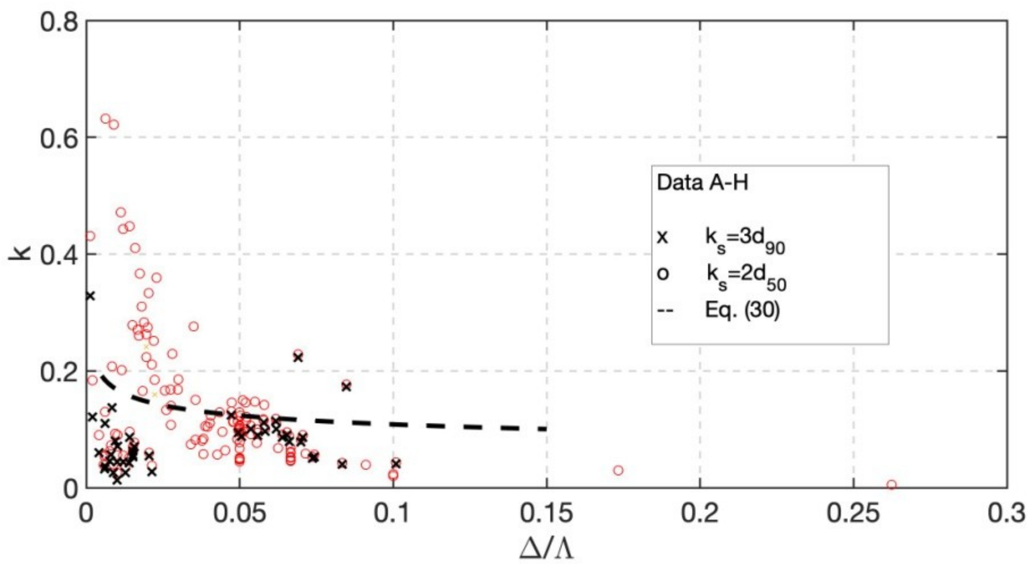


Figure 4.4 - Empirical drag coefficient κ as a function of dune steepness δ = Δ/Λ. Comparison between results obtained by using different equivalent roughness ks (Data A–G see Table 4.1).

The dune steepness may be expressed by means of relative dune height and relative dune length, hence Equation (4.31*)* becomes:

$$\kappa = m \cdot \left(\frac{\Delta}{y} \cdot \frac{y}{\Lambda} \right)^n \tag{4.32}$$

The previous expression is particularly convenient, since according to several authors (Yalin, 1964a; Yalin, 1977; Van Rijn, 1984), the relative dune length may be considered as a constant. Thus, the drag coefficient remains as a function of the relative dune height and, after substituting Equation (4.32) in Equation (4.29), the bed form contribution to the energy slope is:

$$S'' = m\left(\frac{\Delta}{y}\right)^{n} F^{2}\left(\frac{y}{\Lambda}\right)^{1+n} \Gamma_{\Delta/y} \tag{4.33}$$

therefore, once the constant value for the relative dune length has been defined, the energy slope related to the dune drag results as a function only of the Froude number and of the relative dune height.

Introducing a constant value for the relative dune length Λ/y, the best fitting coefficients m and n may change from the previous values (m = 0.053 and n = −0.20) as will be discussed here in the model validation section.

4.6 Model validation and sensitivity analysis to the bed form geometry and skin roughness

In order to validate the proposed model a large selection of 491 field datasets is considered (Table 4.2), including those already used for the preliminary calibration of the empirical coefficient κ (Table 4.1). The Table 4.2 dataset refers to seventeen study reaches on five canals in Pakistan ACP-ACOP (data from Mahmood et al., 1979, as reported by Brownlie (1981)), Niobrara River (Colby and Hembxee,1955), Rio Grande (Culbertson et al. 1976, as reported by Brownlie (1981)), 12 American canals in Nebraska, Colorado and Wyoming AMC (Simons 1957, as reported by Brownlie (Brownlie,1981)), Middle Loup River (Hubble and Mateika 1959, as reported by Brownlie (1981)), Atchafalaya (Toffaletti 1968 as reported by Brownlie (1981)).

Table 4.2 - Summary of field data used to validate the model.

Code	River	N	Q (m^3/s)	Y (m)	U (m/s)	S (m/km)	F (-)	d_{50} (mm)	d_{90} (mm)	Δ (m)	Λ (m)
I	Meuse	44	1743.0-1731.0	9.52-8.22	1.57-0.87	0.141-0.138	0.17-0.09	0.650-0.500	2.500-1.030	0.85-0.58	13.42-7.03
L	ACP-ACOP	151	528.68-27.50	4.30-0.76	1.29-0.35	0.271-0.016	0.29-0.10	0.364-0.083	0.466-0.105	-	-
M	Niobrara	40	16.06-5.86	0.59-0.40	1.27-0.62	1.799-1.136	0.54-0.30	0.359-0.212	0.849-0.326	-	-
N	Rio Grande	33	42.19-1.67	1.51-0.39	1.69-0.10	0.800-0.450	0.49-0.04	0.280-0.160	0.417-0.198	-	-
O	AMC	11	29.42-1.22	2.53-0.80	0.79-0.42	0.330 0.058	0.25-0.10	7.000-0.096	1.440-0.331	-	-
P	MID	38	13.62-9.03	0.41-0.25	1.12-0.59	1.572-0.929	0.72-0.32	0.436-0.215	1.264-0.346	-	-
Q	ATC	55	14186.31-1449.78	14.75-6.92	2.03-0.64	0.051-0.014	0.17-0.06	0.303-0.085	0.708-0.169	-	-

The model is validated comparing the observed and the estimated total energy slope (i.e., $S = S' + S''$ Equation (4.7)), on the basis of the hydraulic parameters listed on Tables 4.1 and 4.2.

In order to assess energy slope S, S' is calculated by Equation (4.12) assuming $k_s' = 2{\cdot}d_{50}$, and S'' is calculated using Equations (4.33) and (4.25):

$$S'' = m\left(\frac{\Delta}{y}\right)^{n} F^{2}\left(\frac{y}{\Lambda}\right)^{1+n} \frac{2\frac{\Delta}{2y}}{\left[1-\left(\frac{\Delta}{2y}\right)^{2}\right]^{2}} \tag{4.34}$$

It results a parametric function of Froude number, dune geometry, and parameters m and n referred to the dune drag coefficient function (i.e., Equation (4.31)):

$$S'' = \varphi\left(F^2, \frac{\Delta}{y}, \frac{\Lambda}{y}, m, n\right) \tag{4.35}$$

Therefore the energy slope related to the bed form drag (S'') in Equation (4.34) involves calculation of few parameters on the right-hand side of Equation (4.34). There are many empirical relationships in literature related to the bed form geometry. In the present work, the following have been considered (Van Rjin (1984) and Karim (1999), Equation (4.36) and Equation (4.37) respectively):

$$\frac{\Delta}{\lambda} = 7.3 \tag{4.36}$$

$$\frac{\Delta}{y} = \left[\frac{(S-S')\left(\frac{\Lambda}{y}\right)^{1.2}}{0.47 \cdot F^2}\right]^{0.73} \tag{4.37}$$

It is worth noting that Equation (4.37) gives relative dune height as a function of only hydraulic parameters, and equivalent skin roughness, because of introducing Equations (4.12) and (4.36) in Equation (4.37). Consequently, the best fitting parameters in Equation (4.33) result:

$$m = 0.07 \quad n = -0.19 \tag{4.38}$$

which are slightly different from those preliminary assessed (m = 0.053, n = 0.20). Figure 4.5 shows the comparison between calculated and measured energy slope.

Accounting for the observed water depth y and Froude number F (see column 5 and 8 in Tables 4.1 and 2), the following equations are involved in energy slope calculation: S' is calculated with Equation (4.12) assuming $k_s' = 2 \cdot d_{50}$ (see column 9 in Tables 4.1 and 2); S'' is calculated with Equation (4.34) along with parameters m and n from Equation (4.38), and eventually bed form steepness using Equations (4.36) and (4.37).

It is worth noting that in the range of observed mean flow velocity U, ranging 0.5–2.0 m/s, the skin roughness contribution gives a shear velocity $u_*' = (g\ S'\ y)^{0.5}$ in the range of about 0.02–0.06 m/s, and the dimensionless conveyance coefficient $C = (g\ S\ y)^{0.5}$ respectively correspond to approximately 7–14 and 22–27.

Among the field data, 93.5% lies within the ±30% discrepancy band error and 68.2% of the measured energy slope lies within the ±20% band error. Considering the large uncertainties in field measurements, this can be considered as a good agreement.

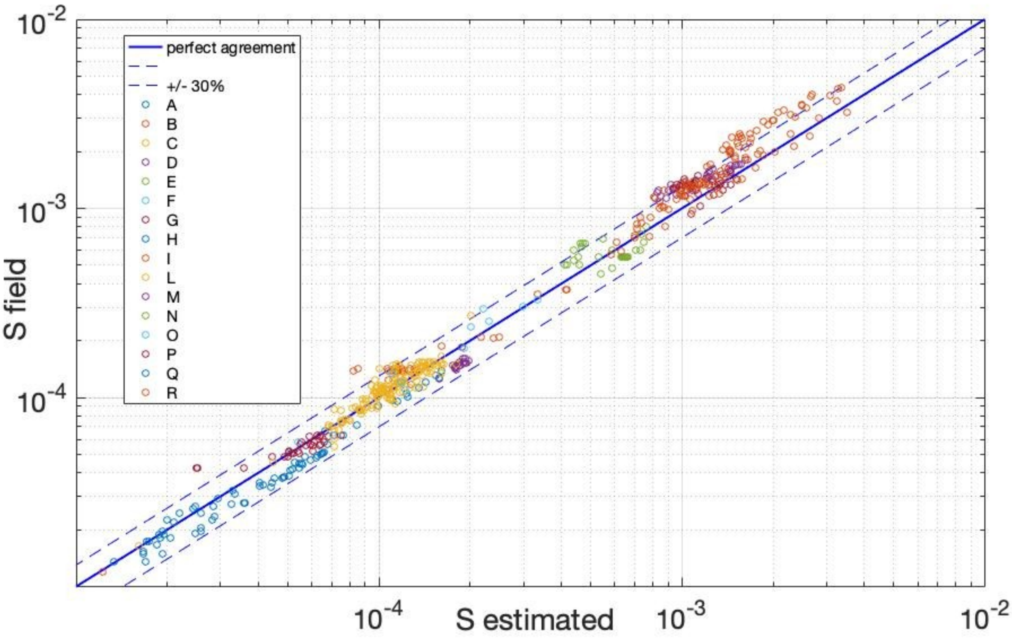


Figure 4.5 - Comparison of estimated and observed total energy slope (the dashed lines represent the ±30% error band).

Skin roughness, as well as the adopted resistance formula, affects the grain flow resistance component and the related energy gradient calculation. Following the linear approach, the contribution due to the bed surface is equal to that of plane bed with identical hydraulic condition and sediment characteristic, without any bed form. The concepts of Karman–Prandtl logarithmic velocity distribution and Nikuradse equivalent grain roughness are considered (Equations (6.9) and (6.10)), where the latter is proportional to the characteristic grain size (Yen, 2002). To test the sensitivity of the model, Nikuradse equivalent roughness $k_s' = 1.0 \cdot d_{50}$ (Keulegan, 1938) and a different approach, reflecting Manning–Strickler Equation (4.39), are considered:

$$n' = 0.0416 \cdot d_{50}{}^{0.165} \tag{4.39}$$

where n' is Manning–Strickler coefficient related to the grain roughness. Hence:

$$S' == F^2 g \left(\frac{n'}{y^{1/6}} \right)^2 \tag{4.40}$$

and Equation (39) is used instead of Equation (4.12) to calculate the energy slope skin roughness contribution when Manning–Strickler formula is considered.

On the other hand, the assessment of bed form drag contribution involves dune geometry, in terms of dune steepness $\delta = \Delta/\Lambda$ or, equivalently, the dimensionless dune height Δ/y and dimensionless dune length Λ/y (see Equation (4.34)). Different approaches proposed by Yalin (1964a, 1977), via empirical consideration based on field data, and reflecting theoretical consideration (Equations (4.41) and (4.42), respectively) were considered to explore the sensitivity of the proposed model to the dune geometry.

$$\Lambda/y = 5 \tag{4.41}$$

$$\Lambda/y = 6.28 \tag{4.42}$$

Table 3 reports the results of sensitivity analysis. Decreasing the value of *Λ/y* leads to a decreasing of the model accuracy. In fact, the field data included within the ±30% error band decrease from 93.5% to 91.3%, whereas the data included within the ±20% error band remains almost stable. By reducing the Nikuradse equivalent roughness (i.e., $k_s' = 1\ d_{50}$), field data included within the ±30% error band reduce from 93.5% to 90.7%, and the data included within the ±20% error band decrease from 68.4% to 65.5%. On the contrary, using the Manning–Strickler resistance formula (Equation (4.39)) only 88.2% of the predicted energy slope are within the ±30% band error and the data included within the ±20% band error are reduced to 65.5% (see Figure 4.6).

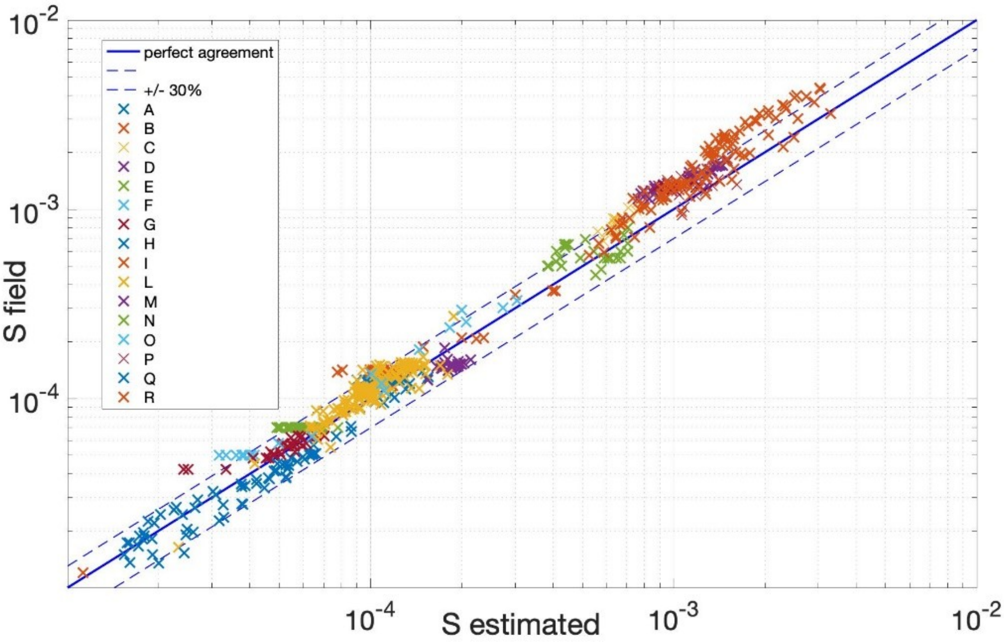


Figure 4.6 - Comparison of estimated and observed total energy slope (the dashed lines represent the ±30% error band). Note: the total energy slope S is calculated accounting for the grain component S' obtained by Manning–Strickler Equation (4.40).

Table 4.3- . Sensitivity analysis of the bed surface roughness and the dune geometry predictor.

Dataset	Λ/y	S'	Validated Data within Error Band	
			30%	20%
model	7.30	*Equation (6.11)* $k_s' = 2.0 \cdot d_{50}$	93.5%	68.4%
test	6.28		93.1%	69.4%
test	5.00		91.3%	68.6%
test	7.30	*Equation (6.11)* $k_s' = 1.0 \cdot d_{50}$	90.7%	62.5%
test		*Equation (6.38)* $n' = 0.0416 \cdot {d_{50}}^{0.165}$	88.2%	63.9%

4.7 Conclusions

The focus of the present paper is the bed form contribution to the flow resistance in natural sand-bed rivers with dune bed forms, assuming a linear separation approach between skin roughness and dune bed form drag. To this aim, the momentum balance and the energy balance equations are applied to 2D flow in open channel, assuming hydrostatic pressure distribution over the cross sections bounding the control volume, which includes a reference bed form pattern. The related energy loss deviates from that derived by Borda–Carnot's applied theorem. The resulting equation in terms of energy grade takes into account an empirical correction factor due to the actual flow field and bed form pattern. The empirical coefficient results as a power function of the dune steepness and is slightly dependent on the Nikuradse equivalent roughness of the grains. Thus, the dune contribution to the energy slope remains an explicit function of dune geometry, in terms of relative length and height, as well as Froude number. The model has been validated using 491 field measurements referred to sand rivers in presence of dunes, showing a good agreement. A sensitivity analysis on estimated energy slope was carried out in terms of dune geometry and the skin roughness adopted model. In particular, different relative dune length approaches were considered. Decreasing the relative dune length more than 30% of the reference value does not substantially degrade the model accuracy, whereas the model is relatively more sensitive to the Nikuradse equivalent roughness, and to the adopted resistance formula (e.g., Manning–Strickler formula).

5. Moving dunes constrain flow hydraulics in mobile sand-bed streams: the Fiumi Uniti and Savio river cases (Italy)

5.1 Introduction

River systems are the main natural network for sediment transfer. They transport sediment from sources to storage areas, i.e. floodplains, lakes and, above all, coasts. The coastal environment is highly influenced by fluvial processes, especially in terms of the balance between the quantity and variability of sediment input and the amount redistributed along shore and offshore by wave action. In this regard, river sediment supply is a main factor affecting coastline morphological changes over time (Rosati, 2005). Areas of river-coast sediment exchange have typically been subjected to extensive anthropogenic activities including: port construction (Kudale, 2010; Tsoukala et al., 2015), mineral and aggregate deposits exploitation (Padmalal et al., 2008), urbanization, dredging to maintain or develop navigation, modifications of hinterland land use (Imassi and Snoussi, 2003), damming (Inman and Jenkins, 1984; Kondolf, 1997; Poulos and Collins, 2002; Yang, 2005), etc. All these activities have specific impacts that alter the natural equilibrium and make the sustainable and integrated management of fluvial-coastal systems very difficult to achieve. In order to understand the ongoing processes that led to severe beach degradation and instability and to define the baseline conditions for a scientifically based management and prediction of future coastal changes, part of the Adriatic coast in the Emilia-Romagna region (Italy) was identified as a reference study case. This coastal area has undergone marked beach retreat in the last decades and this serious problem is still unresolved, in spite of the very many engineering works and artificial nourishments implemented to stop or, at least, to mitigate it. The Emilia-Romagna coast is well known in Europe as one of the most visited touristic areas of Italy for summer holidays; conservation of this system is therefore crucial for both the ecosystem preservation and the local economy (Armaroli et al., 2006; Armaroli et al., 2012; Perini et al., 2016).
In the last decade, aiming to contrast beach erosion, the Emilia-Romagna Region, which is the local public authority in charge of coastal preservation, authorized repeated nourishments by means of costly exploitation of offshore sand deposits or transferral of sand from Scanno di Goro, a large spit in the southern part of the Po River delta (ARPA 2009, ARPA 2016). The urgent need to remedy or, at least, to mitigate beach erosion resulted in the sand nourishments being implemented without a basic knowledge of the fluvial-coastal system sediment budget; indeed, the river sand supply was the least known and most undefined parameter. With the exception of a few sporadic sediment transport field measurements, bedload flux data at the river mouth during significant floods are scarce and limited to very few rivers (Billi and Salemi, 2004; Billi et al., 2017; Ciavola et al., 2005; Ciavola et al., 2010). Field measurements of bedload are difficult due to limited sampler efficiency and sampling techniques, and also require large human and financial resources. Several bedload transport models have been developed as an easier and more affordable alternative to field measurements. However, the definition of the threshold shear stress for particle entrainment and the roughness contribution of bedforms to flow resistance remain important issues mainly investigated by flume experiments but poorly corroborated by field studies. Nonetheless, resistance to flow in presence of dunes is important for morphodynamic modeling, which represents a strategic tool for examining fluvial geomorphology with as regards multi-objective optimization in water resource management (Bernardi et al., 2013; Bernardi and Schippa,

2014; Bizzi et al., 2015). Most of those models consist of coupled systems of depth-averaged flow mass and momentum equations for the liquid phase and sediment mass balance equation for the solid phase, requiring closure equations related to sediment transport and flow resistance (Cao and Carling, 2002; Wang and Wu, 2004).

The objectives of this study are: 1) to enlarge the existing sediment supply dataset through bedload measurement field campaigns in the coastal reach of two representative rivers (Fiumi Uniti and Savio) of the Emilia-Romagna region; 2) to examine the presence of moving bedforms (by bathymetric surveys) and to understand the influence of bedforms on flow resistance and the hydrodynamic field, which in turn affects sediment transport; 4) to compare the effectiveness of two models, available in the literature, using field data, to assess the total dimensionless bed shear stress as a function of grain and bedform roughness.

5.2 Study site

Fiumi Uniti and the Savio are two small river systems located in the south-eastern part of the Emilia-Romagna region. They both originate from the northern Apennines and reach the Adriatic Sea crossing the coastal plain of Ravenna (Figure 5.1). The Fiumi Uniti river, with about 1000 km^2 of drainage area, is the result of the man-made unification of the lower reaches of the Montone (441 km^2) and Ronco (525 km^2) rivers near Ravenna. The Fiumi Uniti river enters the Adriatic Sea between Lido Adriano and Lido di Dante (Figure 5.1). The Savio river basin, with its catchment area of 647 km2, is located a short distance to the south of the Fiumi Uniti and flows into the Adriatic Sea between Lido di Classe and Lido di Savio (Figure 5.1).

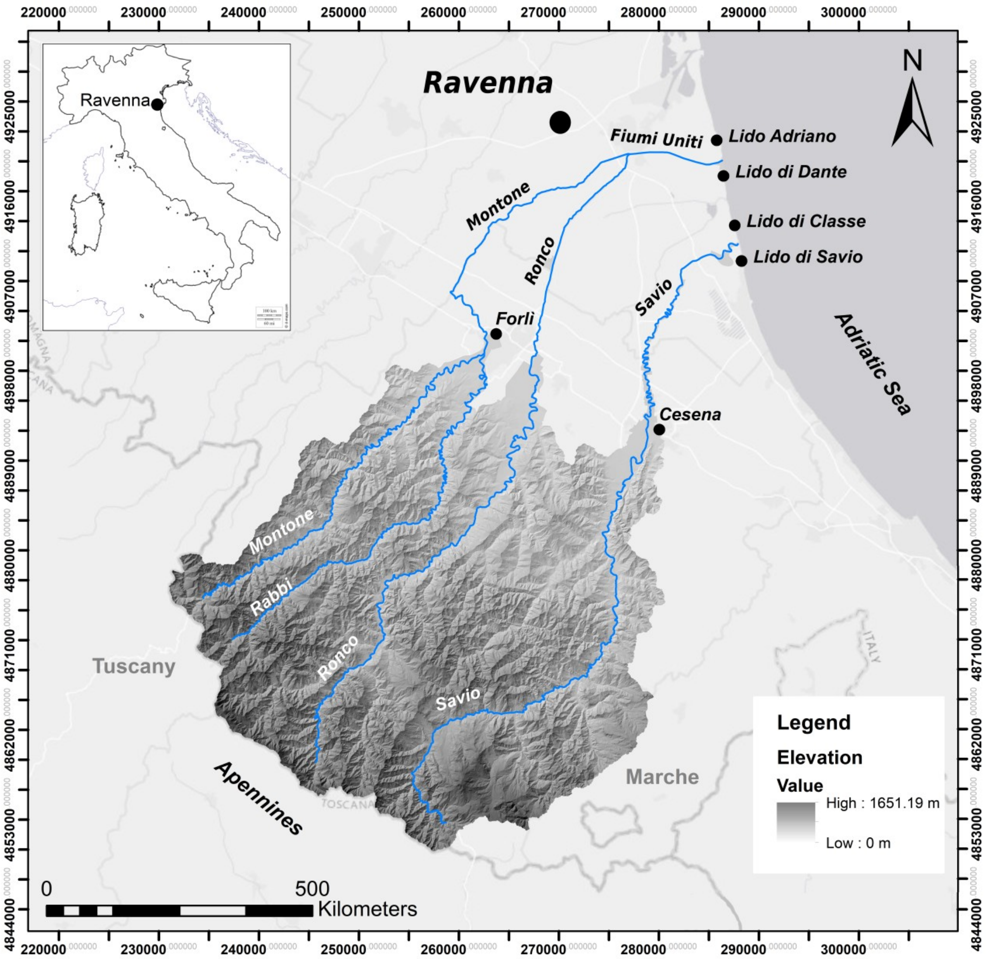


Figure 5.1 - Location map of the Fiumi Uniti and Savio rivers.

In the headwaters, both rivers are underlain by Miocene turbidities (sandstones and marlstones); whereas the alluvial plain mainly consists of Pliocene marine deposits and Quaternary Po river deposits (Amorosi et al., 2002). Both catchment areas are subjected to a Mediterranean climate with a dry summer and precipitation peaks occurring in March, October and November (Mennella, 1972). Annual precipitation is 1060 and 1064 mm for Fiumi Uniti and Savio catchments, respectively. The mean annual temperature is around 13.5 °C for both basins.
In the downstream reach of both rivers a few dams and hydraulic structures are present for irrigation purposes and to prevent salt-water intrusion. The most influential is the Rasponi sluice gate, located 3.5 km upstream the Fiumi Uniti river outlet (Figure 5.3). All these dams are principally used for agricultural purposes and the local Land Reclamation Authority controls their opening and closing during the flooding season (i.e. from October to March) and the dry period (from April to September), respectively.
The sediment transport monitoring sites are located at pedestrian and road bridges located 8 km (Fiumi Uniti) and 3.5 km (Savio) upstream of the river outlet, respectively (Figure 5.22). At the measuring site both rivers have a straight channel with rectangular cross-sections (Figure 5.2). The stream bed gradient is about 0.0003 m/m for both the Fiumi Uniti and the Savio. Bed material is sand, with a mean diameter D_{50} of 0.43 mm (Fiumi Uniti) and D_{50} equal to 0.26 mm (Savio).

The tidal excursion of the Adriatic Sea along the study coast is limited to 0.7. m. Although the tidal effect is almost negligible in both rivers, backwater effects are present in case of low water discharges but are irrelevant during floods.

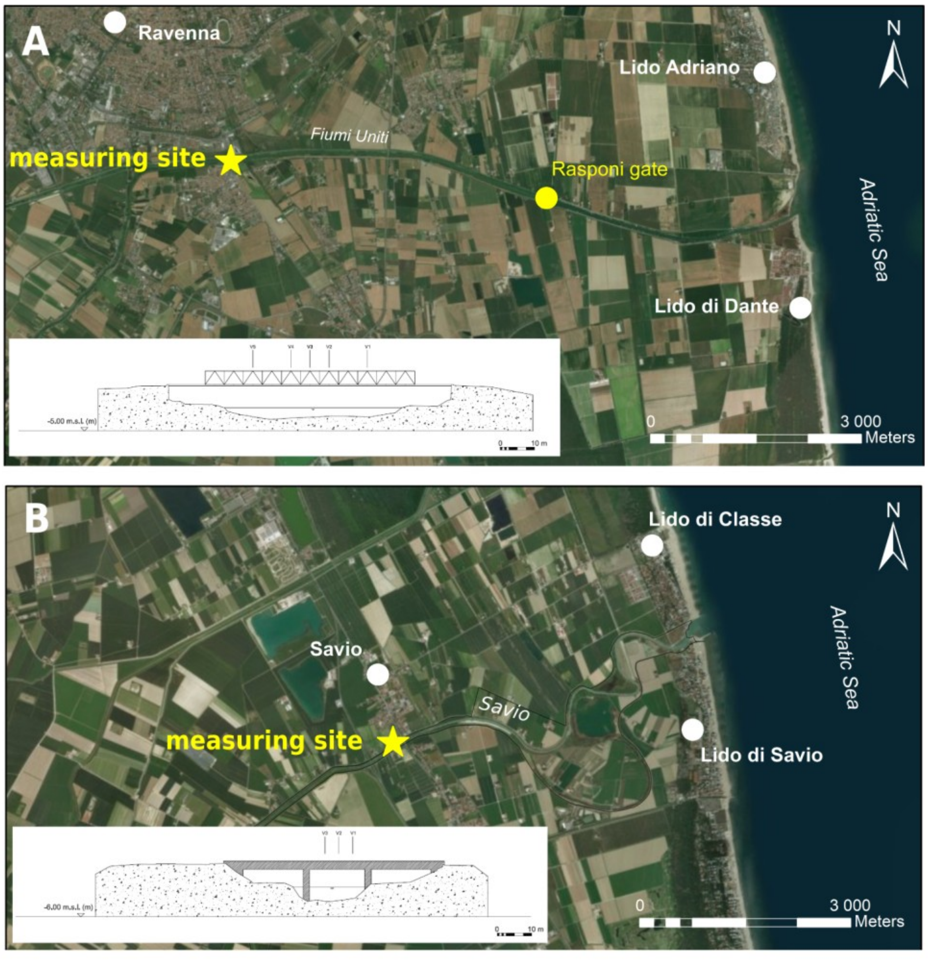


Figure 5.2 - Measuring sites cross sections on Fiumi Uniti (A) and Savio (B).

5.3 Field methods

Field measurements of bedload transport, hydrodynamics and morphological changes were carried out on the Fiumi Uniti in 2005-06 (Billi et al., 2017). A new measuring campaign on both rivers started in 2017 and it is still ongoing. During floods, hydraulic and sediment transport data were collected along fixed verticals. Due to the short time of concentration and flood duration, it was not possible to monitor the entire flood wave of a few floods. Bedforms and morphological changes were surveyed a few days after some selected floods, representative of different flood intensity.

5.3.1 Field measurements

Hydraulic and bedload transport data were collected at fixed verticals (five and three on the Fiumi Uniti and the Savio river, respectively), equally spaced across the active portion of the streambed (Figure 5.2). Flow velocity was measured by a standard USGS AA type current meter and water discharge was calculated by the velocity-area method.

Bedload transport was sampled by a standard Helley-Smith bedload sampler (US BL-84) with a 76x76 mm intake, 0.1 mm of bag mesh and an expansion rate of 1.10, which is considered to provide the highest efficiency (Emmet, 1979). Measurements were taken from the bridges at variable time intervals according to the rate of water level changes detected by a staff gauge installed on the bridge. Since the sampling time largely depends on the flow conditions, it was calibrated after a few attempts: in case of high floods and increased sediment transport rates the sampling duration was 10 minutes; an extension to 20 minutes was necessary for low transport rates in order to ensure the collection of a sufficient sample volume (Boiten, 2003).

Grain size distribution was obtained for each bedload sample collected. Each sample was cleared of vegetation debris and other alien materials and then dry-sieved through a standard Ro-Tap shaker with sieves arranged on a ½ phi scale.

5.3.2 Hydrodynamic model

An integrated approach involving GIS and a quasi steady flow routing model (i.e. HEC-RAS) were used to determine the hydrodynamic condition along the river (including the shear stress acting on the bed), accounting for the backwater effects due to the presence of the movable sluice gate dams. In total 31 (for the Fiumi Uniti) and 20 (for the Savio) surveyed cross sections supported by the Italian National Hydrographic Service are available (spaced about 300 m between each other). Using a DEM derived from GIS, the surveyed river cross sections were interpolated to obtain a relative distance of about 10 m. Boundary conditions were the observed flood hydrograph (upstream) and the recorded sea level (downstream). The only hydraulic structure located along the considered reach is the Rasponi's sluice gate dams on the Fiumi Uniti. It was modeled as an internal boundary condition accounting for the actual operation rule. The model was calibrated using the observed water level for 23 (FU) and 11 (Savio) representative flood events in the recent past. According to the actual geometry and to the roughness distribution along the wetted perimeter, the representative Manning's coefficient results $n=0.019\ m^{-1/3}s$ and $n=0.029\ m^{-1/3}s$ for the Fiumi Uniti and Savio, respectively.

5.3.3 Bathymetric survey

Bathymetric data were collected during low discharge intervals between floods. Data were collected from a tender equipped with an Ohmex SonarMite V1 single beam eco-sounder (2 Hz). The bathymetric sounder was coupled with a DGPS (in RTK correction). The instrument performed continuous recordings with ±0.05 m of accuracy (planimetric and vertical). Measurements were carried out both in zigzag and longitudinal tracks. The longitudinal tracks were parallel to the banks, whereas the zigzag tracks were carried out along a line making an angle of 45° with the thalweg. The longitudinal transects were at 0.25, 0.50 and 0.75 width distance from one bank. The boat velocity was maintained constant, about 2 m/s, in order to ensure the stability of the vessel and the accuracy of the measurements. The reaches surveyed were located upstream of the measuring sites and were about 1 km long (Figure 5.3). The bathymetric measurements were performed on December 6, 2017 and on March 28, 2018 on the Fiumi Uniti and on November 22, 2017 and April 11, 2018 on the Savio.

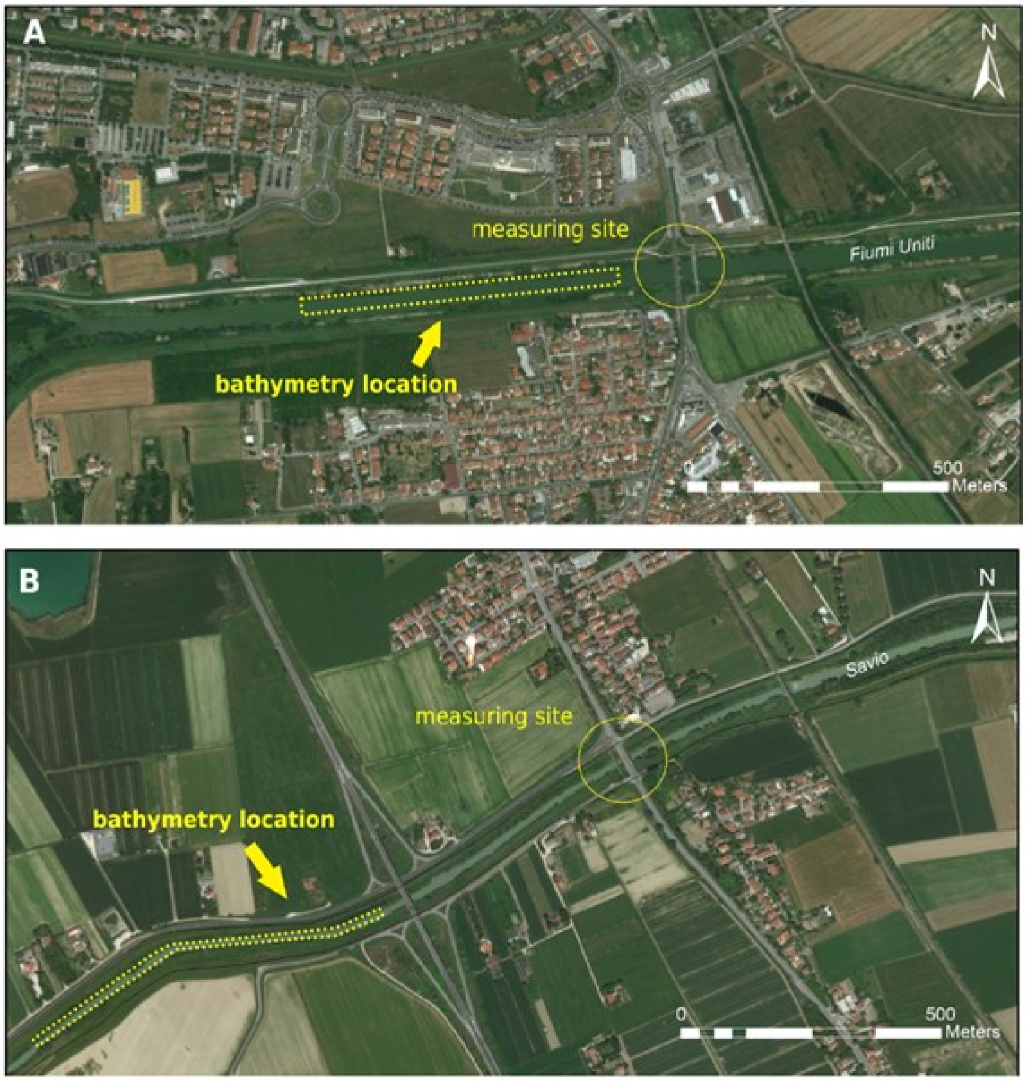


Figure 5.3 - Bathymetry locations of the Fiumi Uniti (A) and Savio (B).

The bathymetric data were post-processed with a moving average filter and interpolated with ArcGIS software through a topo-to-raster interpolation with a 0.5 m grid. The interpolated topographic surfaces were used to evaluate the presence of bedforms, which were assumed as immobile at low-base flow. The data collected enabled to calculate the average dune height and wavelength.

5.3.4. Bedform roughness assessment

In straight alluvial rivers with sediment transport, flow resistance is generated by several factors, of which the most important are boundary surface and bedform roughness, acting as micro and macroroughness respectively. In a fully turbulent flow, the boundary roughness depends on the ratio between flow depth and grain size, whereas the drag form resistance is related to the eddy formations and secondary circulations over the bedform, i.e. to the sudden flow expansion beyond the dune crest (Engelund 1966). Although alluvial river bed roughness has been extensively investigated by many authors (e.g. Einstein and Barbarossa, 1952; Simons and Richardson, 1966; Engelund, 1966; Engelund and Hansen, 1967; Van Rijn, 1982; Yang et al., 2005), these models frequently provide inaccurate predictions of flow resistance; moreover, very little is known about the roughness effect of moving bedforms in natural streams. In this study, two models were tested using our field data: Engelund and Hansen (1967) and Schippa et al. (2019). Both methods are based on a linear approach, originally introduced by Meyer-Peter and Mueller (1948) and Einstein and Barbarossa (1952), which considers an overlapping of skin friction and form drag effects in terms of shear stress. The two components (τ' and τ'' respectively) are usually assumed independent of each other, as showed in Equation 5.1:

$$\tau = \tau' + \tau'' \tag{5.1}$$

The flow separation downstream of the dune crest produces a large turbulence area resulting in energy loss. Disregarding the loss of energy caused by the gradual contraction of flow along the stoss side, the dune contribution to flow resistance reduces to the sudden flow expansion downstream from the dune crest. Engelund and Hansen (1967) applied the Carnot-Borda theorem, which holds strictly in case of pipe flow, whereas Schippa et al. (2019) considered free surface flow, introducing an empirical drag form coefficient (κ_δ) function of dune steepness ($\delta = \Delta/\Lambda$, where Δ and Λ are the dune height and dune length respectively). In terms of energy losses, the dune drag results as in Equation 5.2:

$$\Delta H'' = \kappa_{\Delta/\Lambda} \frac{q^2}{gy^2} \Gamma_{\Delta/y} \tag{5.2}$$

where q is water discharge per unit width of the channel, y is the flow depth, g is the gravity and $\Gamma_{\Delta/y}$ is a function of relative depth of the dune (Δ/y) (see Table 5.1). The form drag contribution may also be related to the energy slope by means of the Froude number $F=U/(gy)^{0.5}$, where U is mean flow velocity:

$$S'' = \frac{\Delta H''}{\Lambda} = \kappa_{\Delta/\Lambda} \cdot F^2 \frac{y}{\Lambda} \cdot \Gamma_{\Delta/y} \tag{5.3}$$

The representative relations of the two approaches are summarized in Table 5.1.

Table 5.1 - Characteristic equations of the two methods used to calculate the grain and bedform roughness considered in this study: Engelund and Hansen (1967) and Schippa et al. (2019).

Engelund and Hansen, 1967 **Engelund & Fredsoe, 1982**		**Schippa et al. , 2019**	
$\tau_* = \frac{\gamma HS}{(\gamma_s - \gamma) d}$	(5.4)	$\tau_* = \frac{\gamma H\, S}{(\gamma_s - \gamma) d}$	(5.11)
$\tau_*' = \frac{\gamma H' S}{(\gamma_s - \gamma) d}$	(5.5)	$\tau_*' = \frac{\gamma H\, S'}{(\gamma_s - \gamma) d}$	(5.12)
$\tau_*'' = \tau_* - \tau_*'$	(5.6)	$\tau_*'' = \frac{\gamma H\, S''}{(\gamma_s - \gamma) d}$	(5.13)
$\frac{V}{\sqrt{g\, H' S}} = 6 + 2.5 \ln \frac{H'}{k_s'}$	(5.7)	$S' = \frac{Fr^2}{\left[2.5 ln\left(11 \frac{H}{k_s'}\right)\right]^2}$	(5.14)
$k_s' = 2.5 d_{50}$	(5.8)	$S'' = k_\delta Fr^2 \left(\frac{H}{\Lambda}\right) \Gamma_{\frac{\Delta}{h}}$	(5.15)
lower regime ($\tau_*' < 0.55$):		$k_s' = 2 d_{50}$	(5.16)
$\tau_*' = 0.06 + 0.4\, \tau_*^2$	(5.9)	$k_\delta = 0.05 (\delta)^{-0.5}$	(5.17)
$\tau_*' = 0.06 + 0.3\, \tau_*^{3/2}$	(5.10)	$\Gamma_{\frac{\Delta}{h}} = \frac{2\left(\frac{\Delta}{2H}\right)}{\left[1 - \left(\frac{1\Delta}{2H}\right)^2\right]^2}$	(5.18)
		$\frac{\Delta}{H} = \left[\frac{\left\{S - 0.0168\left(\frac{d_{50}}{H}\right)^{0.33} Fr^2\right\}\left(\frac{\Lambda}{H}\right)^{1.2}}{0.47 Fr^2}\right]^{0.73}$ (Karim, 1999)	(5.19)
		$\Lambda = 7.3H$ (Yalin, 1964a)	(5.20)

R' is the hydraulic radius related to grains; S is the total energy gradient slope; S' and S'' are the energy gradient slopes related to grains and bedforms, respectively; s is the relative density of the sediment grains equal to ρ_s / ρ_w; d is the representative diameter of the grain (d_{50} is used), Δ is the bedform height, Λ is the wave length of the bedform, $Fr = U / \sqrt{gH}$ is the Froude number where U is the mean flow velocity, H is the mean water depth and g the gravity acceleration. k_s' is the equivalent roughness related to grain and k_δ is the dune drag coefficient.

Eq.(5.9) refers to Engelund & Hansen (1967) method.
Eq.(5.10) refers to Engelund & Fredsoe (1982) method.

5.5 Results

5.5.1 Flow conditions, bed material characteristics and dune geometry

Discharge variations in the Fiumi Uniti and Savio rivers during the period 2017-2018 are reported in Figure 5.4, in which the field measurements are also reported.
Flow discharge of the monitored floods ranged from 13 to 358 m^3/s in the Fiumi Uniti and from 7 to 132 m^3/s in the Savio. The 358 m^3/s flood was one of the highest ever recorded in the last decade (Billi et al., 2017). Flow velocity varied widely between 0.2 and 1.66 m/s (Fiumi Uniti) and from 0.21 to 1.50 m/s (Savio). Mean flow depth varied between 1.3 and

4.7 m in the Fiumi Uniti and from 1.99 to 3.96 m in the Savio. Mean water slope, obtained from the model, was 0.00073 and 0.00017 m/m for Fiumi Uniti and Savio, respectively. Figure 5 shows the composite grain size frequency distribution curves of bedload samples. Average bedload D_{50} is medium to coarse sand: 0.51 mm (Fiumi Uniti) and 0.48 mm (Savio). No significant change of D_{50} with water discharge was observed.

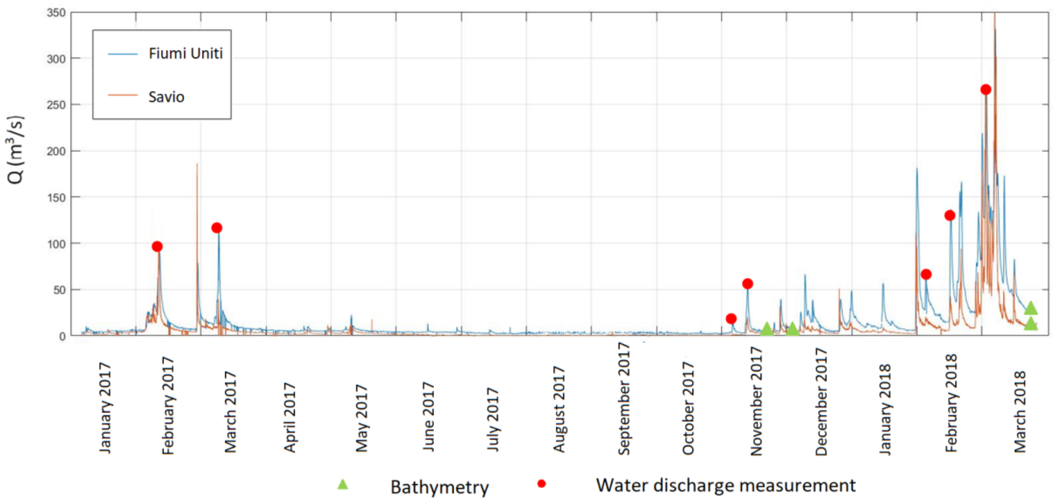


Figure 5.4 - Hydrograph of the Fiumi Uniti and Savio from March 2017 to March 2018 (data sources: Emilia-Romagna Regional Agency for Prevention, Environment and Energy (Arpae) and Italian Hydrographic Service).

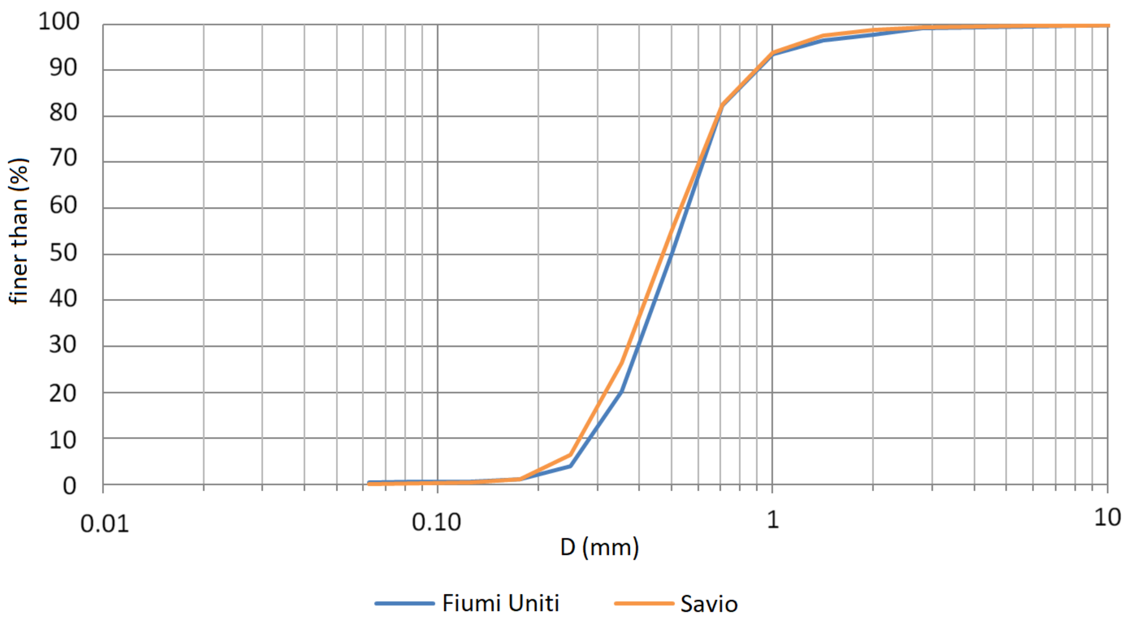


Figure 5.5 - Composite grain size distribution of the Fiumi Uniti and Savio bedload samples.

The monitored floods occurred on April 14, 2005 (Billi et al., 2017), on November 14, 2017 and March 12, 2018. In the last two cases mean cross-sectional velocity varied between 0.84 and 1.34 m/s in the Fiumi Uniti and between 0.61 and 1.50 m/s in the Savio (Table 5.2). Table 5.2 reports data including the highest monitored flood, i.e. the one of 358 m^3/s recorded in 2005 from Billi et al. (2017). The bathymetric data were interpolated in ArcGIS, obtaining a 0.5-m grid and smoothed profiles were extrapolated to obtain the dune geometry (Figure 5. 6).

Post-processing analysis returned dune heights between 0.10 and 0.28 m (Fiumi Uniti) and between 0.12 and 0.16 m (Savio). Mean dune wavelength was 15.41 m in the Fiumi

Uniti and 6.65 m in the Savio. Dune steepness is about 0.007 in the Fiumi Uniti (except for the 2005 flood, which is about 0.02) and 0.02 in the Savio. Data are reported in Table 5.3.

Table 5.2 -- Mean cross sectional hydraulic parameters and grain size

Date	River	Flow discharge [m^3/s]	Mean flow velocity [m/s]	Water depth [m]	Energy gradient slope [m/m]	Bedload D_{50} [mm]	Bedload [ton/day]
12/04/2005*	Fiumi Uniti	358	1.66	4.72	0.000127	0.42	510.88
14/11/2017	Fiumi Uniti	95.90	0.84	2.52	0.000051	0.57	1.43
12/03/2018	Fiumi Uniti	291	1.34	4.09	0.00014	0.47	297
14/11/2017	Savio	22	0.61	2.30	0.00012	0. 48	0.03
12/03/2018	Savio	132	1.50	3.96	0.00035	0.51	189.69

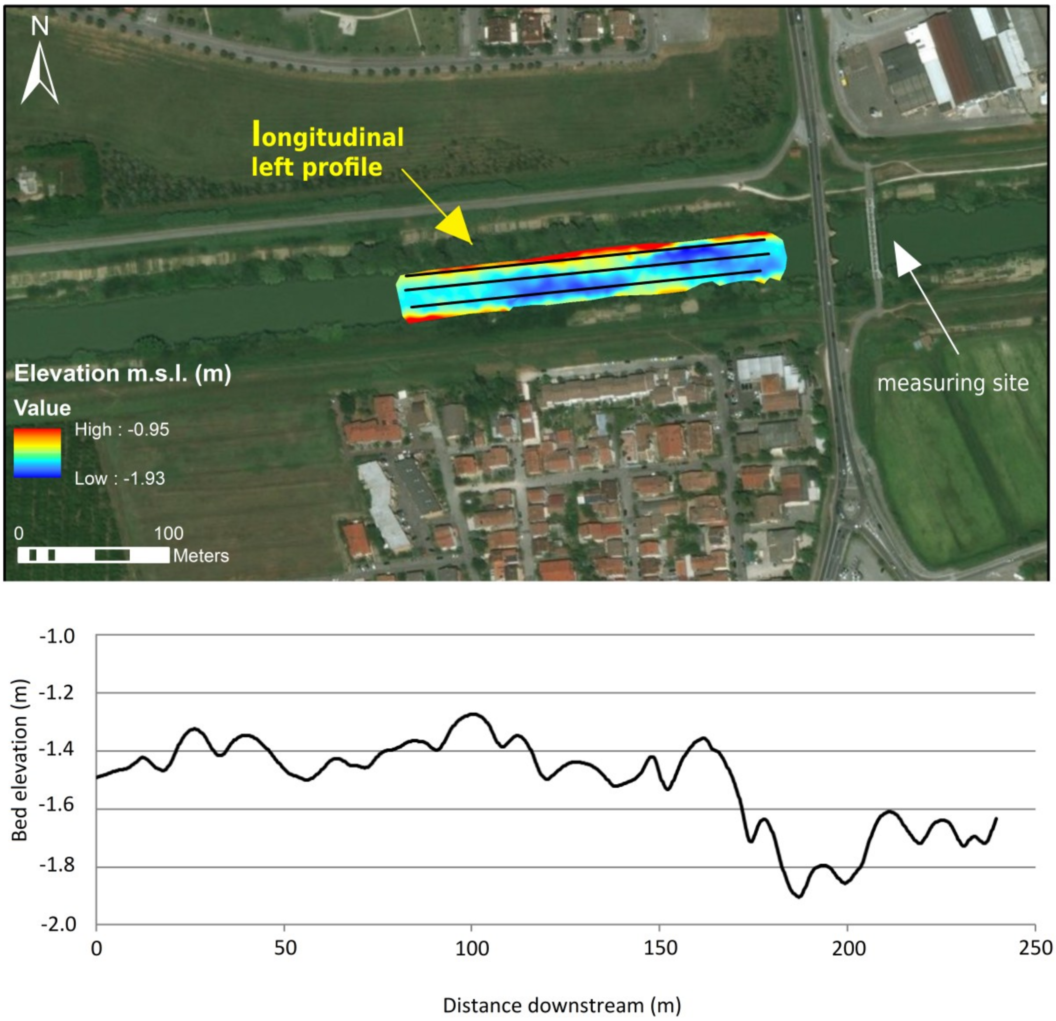


Figure 5.6 - Longitudinal left profile of the interpolated topographic surface obtained from the bathymetric survey of the Fiumi Uniti taken on November 14, 2017.

Table 5.3 - Bedform geometry parameters of Fiumi Uniti and Savio rivers

Date	River	Flow discharge [m³/s]	Fr	Re*	*ff*	Δ[m]	*Λ* [m]	Δ/ *Λ*
12/04/2005*	Fiumi Uniti	358	0.24	32.41	0.02	0.28	13.10	0.02
14/11/2017	Fiumi Uniti	95.90	0.17	20.24	0.01	0.10	17.53	0.006
12/03/2018	Fiumi Uniti	291	0.21	35.11	0.02	0.13	15.61	0.008
14/11/2017	Savio	22	0.14	23.87	0.06	0.12	6.14	0.020
12/03/2018	Savio	132	0.25	56.32	0.05	0.16	7.16	0.022

In the present table: $Fr=v/(gR)^{0.5}$ where g is gravity, R the hydraulic radius, and v mean flow velocity is the Froude number, being ; Re*is the dimensionless Reynolds number defined as $(u_* d)/\nu$, $u_* = \tau/\rho$ is shear velocity, d a representative grain diameter (normally D_{50}) and ν the cinematic viscosity; *ff* is Darcy-Weisbach friction factor; *Δ* is the bedform height, *Λ* is the bedform wave length and *Δ/ Λ* is the dune steepness. (*) Billi et al. (2017) data.

5.5.2 Roughness calculation

Dimensionless total shear stress was obtained applying the approach of Engelund and Hansen (1967), later modified by Engelund and Fredsoe (1982) (Figure 5.7) and Schippa et al. (2019), (Figure 5.7).

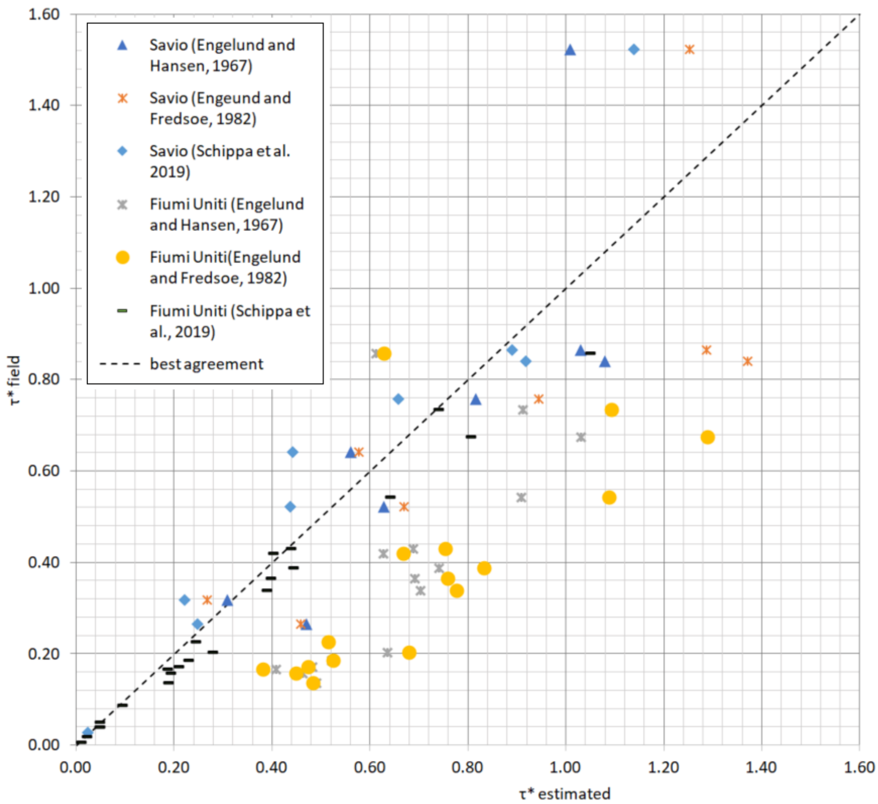


Figure 5.7 - Relation between calculated and field values of dimensionless shear stress (τ_*), using the approach of Engelund and Hansen (1967), Engelund & Fredsoe (1982) and Schippa et al. (2019), (see Table 5.1).

Figure 5.8 shows the relation between τ^* and $\tau^{*\prime}$ calculated with the methods of Engelund and Hansen (1967), Engelund & Fredsoe (1982) and Schippa et al. (2019), for the Fiumi Uniti (Figure 5.8A) and Savio (Figure 5.8B), respectively. Figure 5.9 gives an overview of the relative contribution made by the bedform drag and the skin roughness to the total shear stress. Figure 5.10 plots the comparison between the hydraulic grade energy estimated according to the three cited methods, and the observed values.

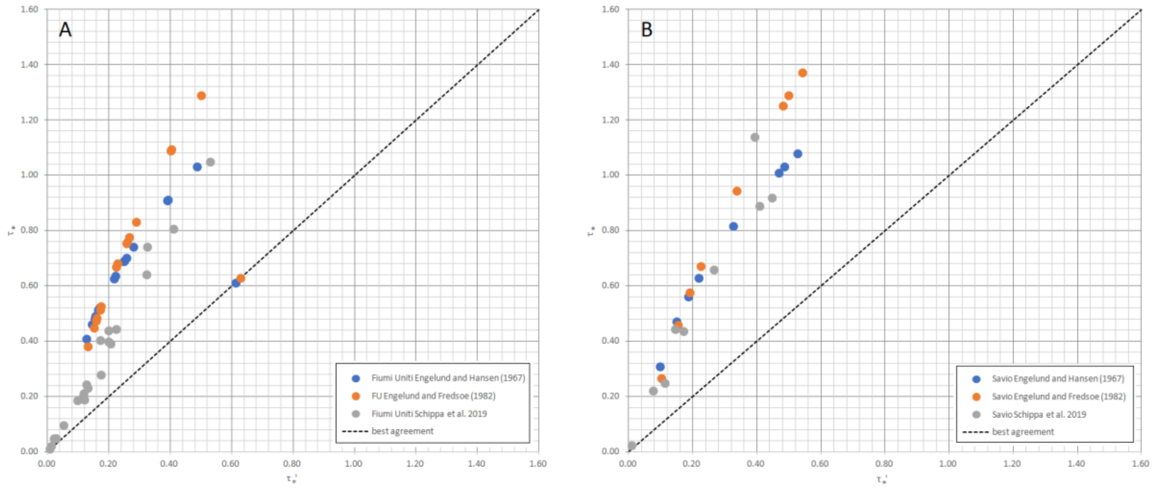


Figure 5.8 – Relation between τ_* and $\tau_*^{\prime}$ of the Fiumi Uniti river (A) and Savio river (B) calculated with Engelund and Hansen (1967), Engelund and Fredsoe (1982) and Schippa et al. (2019) approaches. Engelund and Hansen (1967) approach has been calculated through the simplified equations for lower flow regime with a ripple or dune bed equation (Eq.5.9), Engelund and Fredsoe (1982) approach with Equation 5.10.

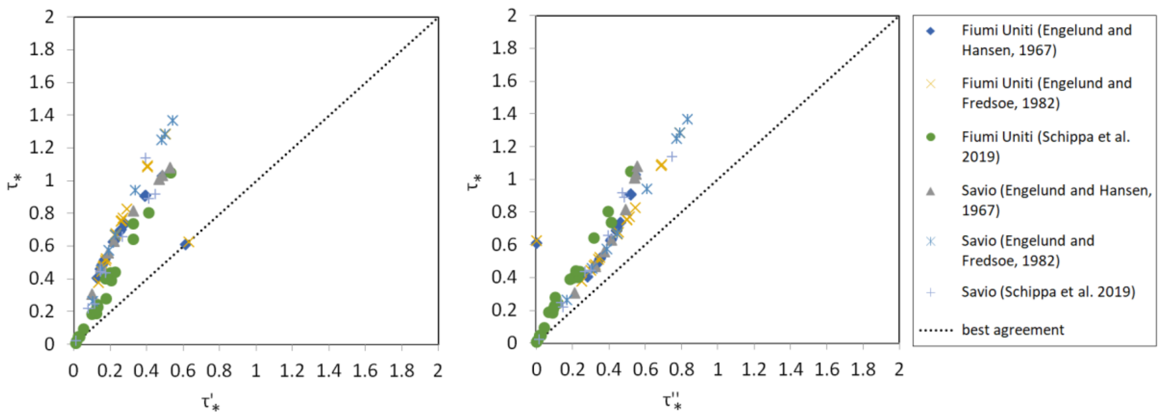


Figure 5.9 - Contribution of bedforms ($\tau^{*\prime\prime}$), and skin roughness ($\tau^{*\prime}$), to the dimensionless total shear stress τ^*, for the Fiumi Uniti and the Savio rivers, calculated with Engelund and Hansen (1967), and Schippa et al. (2019) (see Table 5. 1).

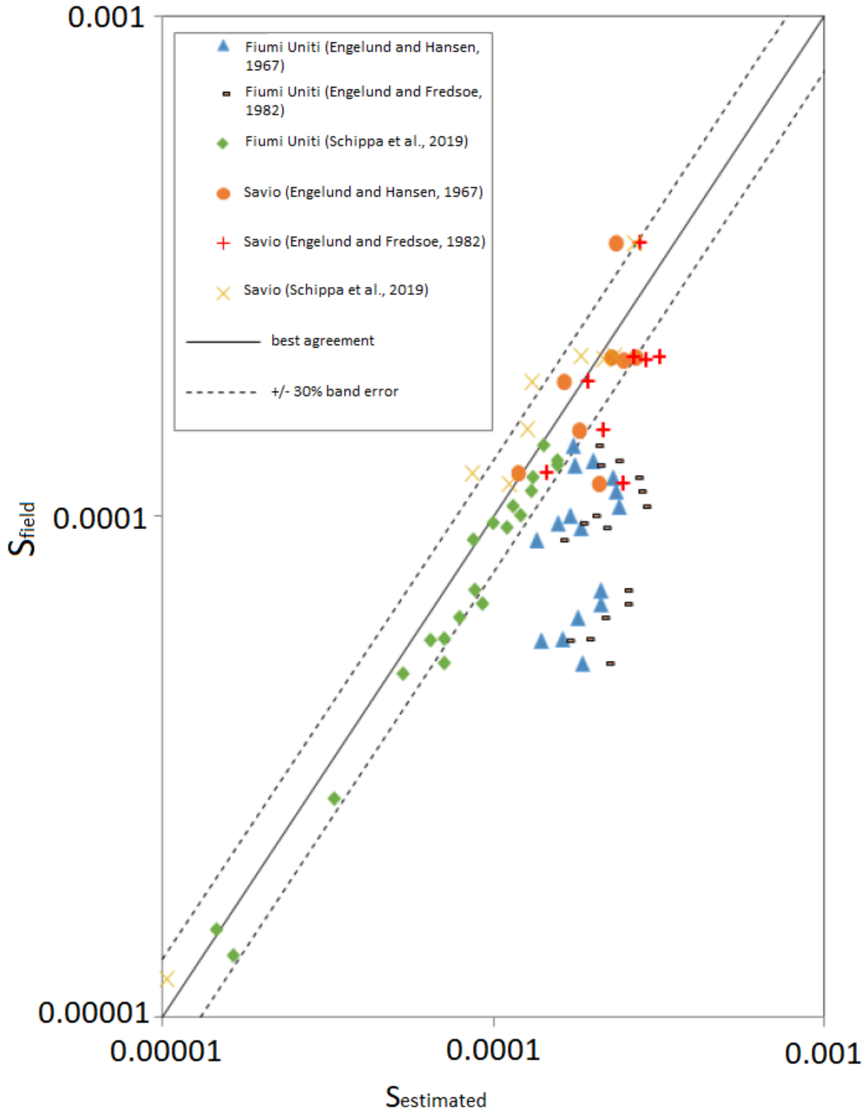


Figure 5.10 - Energy grade observed and estimated using Engelund and Hansen (1967), Engelund and Fredsoe (1982) and Schippa et al. (2019) methods (see Table 5.1).

5.6 Discussion

The field data confirmed the hydrological affinity between the two study rivers, thus providing the basis for replicating the proposed approaches on rivers with similar characteristics. The neighboring catchments, in fact, have parallel geological characteristics and precipitation resulting in a similar flood pattern and grain size distribution of bed material. Both the study rivers have a sand bed and it was found that bedload D_{50} does not change significantly with flow discharge. Although spanning a wide range discharges, all measured floods were characterized by a subcritical flow regime as Froude numbers ranged between 0.14 and 0.25 (Tab.5.3), i.e. conditions typical for the formation of dune bedforms. In our study rivers, however, relative dune height and steepness are smaller than those reported in the literature (Flemming, 1978; Shen et al. 1978; Brownlie, 1981; Kostachuck et al., 1989; Julien, 1992; Gabel, 1993; Prent and Hickin, 2001, Schippa et al., 2016; Schippa, 2020). These authors worked on much larger rivers (all of them with sand beds as in the study rivers of this current investigation - Table 4), in which the average dune height is about 0.36 m and mean steepness is 0.06. As evidenced by Bennet (1997), dune geometry is primarily controlled by flow hydraulics, dune wavelength ranges between 6 to 7 times flow depth (y) and the maximum steepness is around 0.06. In our study rivers, dune wavelength is more variable as it ranges from 2*y* to 7*y*. Bennet (1997) found that the maximum dune height is approximately 0.4*y*, by contrast, our dunes height is one order of magnitude smaller (0.04 - 0.05 *y*). Though our dunes are flatter than those of Bennet (1997), their geometry values are within, or very close to, the range reported in the literature (Flemming, 1978; Shen et al., 1978; Brownlie, 1981; Kostachuck et al., 1989; Julien, 1992; Gabel, 1993; Prent and Hickin, 2001), (Table 5.4), with the sole exception of the dune steepness. The smaller dune steepness measured in the study rivers may be accounted for by the combination of two main factors: the low bedload transport rates and the extremely rapid nature of floods. Commonly, the bedload transport phase of the monitored floods did not last more than 12 hours and, as reported in flume experiments by Corridori (2013), the time for dune bedform development or response to changing hydraulic conditions ranges between 6 and 12 hours. Therefore, we can assume that the combination of a moderate sediment supply with a short time period for full dune development may have resulted in a lower than usual steepness.

Table 5.4 –Literature (see text for reference) fluvial dune geometry average values.

	Δ (m)	Λ (m)	δ	V (m/s)	Q m^3/s	$D_{50}(Q_b)$ mm
Mean	0.36	6.34	0.06	0.64	1072.19	0.35
Max	2.08	54.10	0.094	0.77	6368.00	0.41
Min	0.08	2.00	0.017	0.47	0.82	0.25
St.Dev.	0.31	6.63	0.01	0.07	2043.57	0.04
CV	0.85	1.05	0.24	0.11	1.91	0.11

Corridori's flume experiments (2013) have also shown that, with a decreasing sediment supply, the dune height tends to grow, increasing the total bed roughness. In our study rivers, no evidence of the influence of the sediment transport rate on the dune geometry was found, but that is probably because bedload varies, linearly, with discharge. This hypothesis, however, should be corroborated by other field and flume studies. Julien and Klaassen (1995) concluded that both dune height and wavelength generally increase with water discharge during major floods, whereas dune steepness remains reasonably constant. This conclusion is confirmed by our data, but for quite minor floods of the Fiumi Uniti, probably because of lower sediment feeding. Instead, during higher and relatively longer floods, there is enough time for dunes full development. Nevertheless, since dune measurements were made after the floods, a later reworking of the dunes cannot be excluded. Dune geometry interferes with flow resistance. The Engelund and Hansen (1967) and Engelund and Fredsoe (1982) methods show an overestimation of the observed values, whereas the method of Schippa et al. (2019) returns more accurate results in terms of total dimensionless shear stress τ^* (Figure 5.7) and of total energy grade, S (Figure 5.10). In fact, with the Schippa et al. (2019) method, 84% of the field data lie within the +/- 30% discrepancy error boundaries, whereas Engelund and Hansen (1967) method shows only 30% of predicted data are within the same error boundaries (Figure 5.10). Figure 5.8 reports the dimensionless total shear stress (τ_*) and its component related to the skin friction (τ_*'). Accounting for a fixed skin stress (due to the bed particles) $\tau^{*\prime}$, the bedform component $\tau^{*\prime\prime}$ is higher the in Engelund and Hansen (1967) and Engelund and Fredsoe (1982) approaches. Moreover, subsequent Engelund and Hansen (1967) and Engelund and Fredsoe (1982) corrections show clear difference from the Schippa et a al. (2019) approach. Both Engelund’s approaches, in fact, significantly deviate from the best agreement for values of $\tau^{*\prime}$ higher than 0.2, while, for $\tau^{*\prime}$ values lower than 0.2, the difference between the three methods is less obvious (Fig.5.8). The same evidence is highlighted in Figure 5.9, which compares the relative contributions to the total shear stress due to the bed particles and the bedforms, calculated by the different methods used in this study. The Schippa et al. (2019) method considers dimensionless shear stress due to equivalent contributions from skin and form friction. On the contrary, in the Engelund and Hansen (1967) and Engelund and Fredsoe (1982) methods, almost 60-70% of the total dimensionless shear stress τ^* is attributed to bedforms; thus, the higher the mobility value, the higher the dimensionless shear stress τ^* attributed to bedform. Moreover, it is important to consider the measuring station's proximity to the coast because it is probably subjected to backwater effects, even though the tide range in this part of the Adriatic Sea is very limited (0.2-0.6 m). Though backwater influence was calculated theoretically in this study, its effects may lead to an overestimation of the total energy grade, S (Fig. 5.10).

5.7 Conclusions

A field study to analyse the interaction among bedload transport, bedform patterns and flow resistance was carried out in two representative rivers (Fiumi Uniti and Savio) of the Emilia-Romagna region in Italy. The field campaign also broadened the existing bedload dataset of these rivers in order to quantify the sediment supply to the regional coast. Recognizing the importance of moving bedform presence as a significant and interactive element of control on bedload transport, the investigation also focused on the resistance to flow in presence of bedform. Since 2005, a wide dataset of about 30 floods were measured and the following main results were obtained:

1. Dune height ranged between 0.10 and 0.28 m, wavelength between 6.14 and 17.53 and steepness around 0.02 (except for a few outliners related to Fiumi Uniti river).

2. Comparing hydraulic parameters and bedform geometry, it was possible to classify the bedforms observed in the field as dunes and sometimes small dunes.

3. As referred in the literature data and results, the study rivers' dune height and length of do increase with flow discharge.

4. Dune steepness is reasonably constant (though smaller than the data reported in the literature) and confirms the results of previous studies, for the Savio case. For the Fiumi Uniti river further studies may be necessary.

5. The effect of bedforms on flow resistance was investigated and a semi-empirical approach, based on linear contribution of skin roughness and bedform drag, was tested (Schippa et al., 2019) and compared with the Engelund and Hansen (1967) and Engelund and Fredsoe (1982) approaches. Based on the results, the method recently proposed by Schippa (2019) et al. seems to be more suitable in terms of total dimensionless shear stress (τ_*) estimation.

6. Both Engelund’s approaches significantly deviate from the best agreement for values of $\tau^{*\prime}$ higher than 0.2.

7. For dimensionless shear stress due to skin (τ*’) values lower than 0.2, the difference between the three methods is less evident.
8. According to the linear approach, the field data show an equivalent contribution due to the bedform drag and the skin roughness (as shown by the Schippa et al.,2019 method), whereas Engelund and Hansen (1967) and Engelund and Fredsoe (1982) attribute almost 70% of the total dimensionless shear stress τ* to bedform effect.

9. The measuring site's proximity to the coast and the related backwater effects could be elements of influence in the study of the consecutive overestimation of the total energy grade, S. In order to clarify this point further studies are needed.

The results of the current study, though encouraging, suggest that other methods available in literature should also be tested against field data in order to better estimate the relative amount of roughness provided by grains and bedforms, as well as to investigate the interaction between bedform resistance and flow, bedform geometry and the rate of bedload transport.

6. Evaluation and mid-term reconstruction of bedload transport of two small rivers of the Emilia-Romagna region (Italy) through direct measurements

6.1 Preamble

Bedload transport is one of the most important physical process governing alluvial rivers (Barry et al., 2008). It determines their morphologic development acting dynamically on channel geometry stability reflecting both water and sediment delivered from the watershed (Leopold et al. 1964; Emmett and Wolman 2001). Bedload transport refers to the bed material load which moves as particles rolling, sliding or saltating (contact load) and suspended load, i.e. the particles which are transported and temporarily maintained in suspension in the flow due to turbulent mixing process (Gomez and Church, 1989; Garcia, 1999; Haddachi, 2013). Bedload transport is controlled by the interaction between the quantity and quality (particle size) of upstream sediment supply and the flow pattern (flood frequency and duration) (Turowski et al., 2010; Wang et al., 2011) and its quantification is of paramount importance in many environmental and engineering problems. Over time several approaches and methods have been developed but no one universal method proved to provide reliable results for different kind of rivers. One of the most effective methods consists in direct field sampling, i.e. measuring the bed-load discharge by taking samples from the stream during floods. Despite its technical difficulty, budget constraints (Gaeuman and Jacobson, 2006; Muhammad et al., 2019) and its efficiency strongly correlated with the transport rate variations and the characteristic of bed material (Hubbel, 1987; Gomez, 1991), the field measurement method is the approach which has been preferred for this PhD thesis. One of the most used instrument to measure bedload transport on sand-bed rivers is the Helley-Smith bedload sampler, which was used also for field campaigns of this study. Field measurements are very useful since they permit to obtain reliable data which can be used for model validation and application (Marquis and Roy, 2012). Bedload field measurement and its correlated difficulties and high costs have pushed scientists to develop equations for bedload transport prediction (Schoklitsch, 1934; Bagnold, 1980). Various formulas on bedload transport have been proposed in the past fifty years based on both laboratory and field studies (Habersack and Laronne, 2002), though field measurements are very limited (Molinas and Wu, 2001). One big limitation of bedload transport equation is that they return reliable results only under the specific conditions in which they were developed. Some formulas, in fact, are more appropriate to one type of channel morphology than another (Sidari et al., 2014), requiring a subsequent adaptation of the original formulations to the new condition (Khorram and Ergil, 2010; Haddachi et al., 2013) which is not always returning reliable results.

In this section, a comparison of bedload transport measured in the Fiumi Uniti and Savio river with some of the most well-known bedload transport formulas was done, trying to consider the most appropriate approach to the presented study cases. A restriction to a generalized approach to input variable definition that are easy to implement has been done, obtaining a practical and feasible implementation which can applied in a simple manner over several condition in this rivers of in further studies also from stakeholders for operational purposes.

6.2 Introduction

Knowledge of bedload transport rate is of great interest in many circumstances such as river restoration, ecosystem protection, navigation as well as infrastructure management (Holmes, 2010). In fact bedload transport not only provides the main process linkage between the hydraulic and sediment conditions which govern changes in river morphology, but also make informed management decisions that affect a river's function (Gomez, 2006). Bedload is defined as the coarsest fraction of the river sediment load that moves above the bed by rolling or saltating (Gomez and Church, 1989). Its estimation constitutes one of the main issue for fluvial geomorphologies, sedimentologists, engineers and fluvial managers (Claude et al., 2012). Over time several approaches have been developed to estimate bedload transport capacity but no one universal method proved to provide reliable results for different kind of rivers. Despite all used approaches, one of the most effective consists in direct sampling with mechanical sampler (traps or isokinetic samplers) deposed on the riverbed. These bedload samplers have been widely used in sand-bed rivers (Andrews, 1981; Dietrich and Smith, 1984; Carey, 1985; Van Rijn 1991, 1992; Gaweesh and Van Rijn 1994; Batalla, 1997; Abdel-Fattah et al., 2004; Martin and Ham, 2005; Holmes, 2010; Haddachi et al., 2013) and in gravel ones (Johnson et al., 1977; Andrews, 1994; Ryan and Porth, 1999; Bunte et al., 2004; Liu et al., 2008). Several criticisms have been made on these type of measurements like the fact that they are technically complicated and expensive (Gaeuman and Jacobson, 2006; Muhammad et al., 2019), that they are even dangerous during floods (Gaeuman and Jacobson, 2006) or that they can be also accompanied by errors, which have been estimated to be up to 20 per cent (Wren et al., 2000). Difficulties and inaccuracies in the sampling campaigns have resulted in the development of numerous empirical and physically based formula for bedload transport prediction obtained from laboratory flume experiments. (e.g., duBoys,1879; Schoklitsch, 1934; Bagnold, 1980). These formulae are generally based on idealized hydraulic principles which try to relate bedload transport with water discharge, shear stress, or stream power incorporating (or not) grain size characteristics (Martin, 2003). Despite more than a century of research and the major or minor well-documented limitation of these formulae (Gomez and Church, 1989), bedload modeling remains a challenge (Recking et al.,2016).

The study focuses on two small river systems (Fiumi Uniti and Savio) of the Emilia-Romagna region (Italy) which are pragmatic in term of sediment contribution for the coastal sediment budget alteration. Little is known on contribution of bedload sediment transport by the small rivers of the Emilia-Romagna region (Italy) which, in the last years, has been subjected to alarming coastal erosion processes. In order to face this problem soft mitigation measures as sand replenishment and submerged breakwaters have been used on the coastal zone without solving the root problem. Despite few attempts of bedload measurements have been done in the region, only refers to a few rivers (Billi and Salemi 2004; Ciavola et al., 2005; Ciavola et al., 2010; Preciso et al., 2011; Billi et al., 2017), there is still a lot to investigation in order to define the quantity of sediment supply by local rivers. The objective of this study is the bedload transport estimation contribution to the sea through the implementation of a regional scale project calibrated on the hydrographic context of the north-eastern Apennines of the Emilia-Romagna region. The construction of the model requires the following steps:

1. Definition of the relationship (empirical-experimental) between outflows and bedload transport;
2. Reconstruction of effective bedload transport rate for each significant event, with reference to the available records of the average daily outflows;

3. Statistical analysis of outflows and analysis in the medium to long term prospects of the annual duration curve of the average daily flows (which represent the most widely available data) and the consequent expected bedload contributions.

In this chapter, the attention has been focused on the first two aspects which are considered preliminary elements to the implementation of the model. At the moment the statistical-hydrological analysis has been left in the background. In particular, considering that the hydrological events of flooding are the main causes of the bedload transport contribution to the sea, he contribution given by this study focuses on:

1. the need to correctly estimate the flow of sediments associated with the water flow rate (highlighting how empirical formulas available in literature or based on theoretical transport capacity can provide indications that are very different from the experimental data, as examined for the Fiumi Uniti and Savio cases);
2. the opportunity to reconstruct flood hydrograms starting from recorded daily average flow data, correctly associating bedload transport with water discharge during each events, demonstrating the substantial inconsistency, for hydrographic basins of these characteristics, of a model based on the wrong use of the daily average flow data.

6.3 Study sites

Fiumi Uniti and the Savio river are located in the south-eastern part of the Emilia-Romagna region, Italy. They are two small river systems which originate from the northern Apennines and outflow in the Adriatic Sea near Ravenna province (Fig.1). Their catchment basin are of about 1000 km^2 and 647 km^2 for the Fiumi Uniti and the Savio, respectively. The Fiumi Uniti river, as its name indicates, it the result of the unification of two rivers: the Montone (441 km^2) and Ronco (525 km^2). While the Fiumi Uniti enters the Adriatic Sea between Lido Adriano and Lido di Dante (Figure 6.1), the Savio empties into the Adriatic Sea between Lido di Classe and Lido di Savio (Figure 6.1), placed a few kilometer south of the Fiumi Uniti.

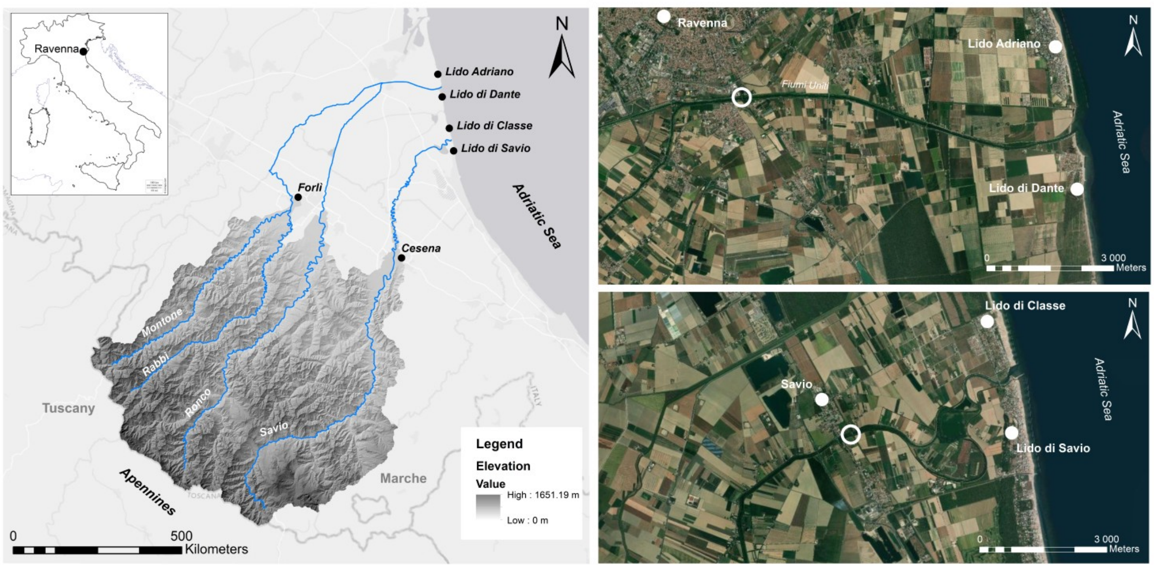


Figure 6.1 - Location map of the study reaches of Fiumi Uniti and Savio rivers. The empty white circle indicates the measuring sites.

In the headwaters, both river basins are underlain by Miocene turbidities, sandstones and marlstones (Figure 6.2). The alluvial plain mainly consists of Pliocene marine deposits and Quaternary Po river deposits (Amorosi et al., 2002). The current land use has similar characteristics over all catchment areas of both basins. While agricultural production and pasture areas occupy nearly 49% of the basin, woody agricultural areas have approximately 2%. Around 46% of the basins are occupied by shrub and forest. Only 1% of the whole basins is attributed to urban zones; remaining 1.8 and 0.2 % are referred to not vegetated areas and water, respectively (Figure 6.2).
The climate is characterized by typical Mediterranean climate with a dry summer and precipitation peaks occurring in March, October and November (Mennella, 1972). The annual precipitation is around 1000 mm both for Fiumi Uniti and Savio catchments. The mean annual temperature is around 13.5 °C for both basins.
In the downstream reach both rivers are characterized by the presence of a few dams which are used for irrigation purposes and to prevent salt-water intrusion which are maintained constantly open during the flooding season (i.e. from October to March) and closed in dry ones period (from April to September).

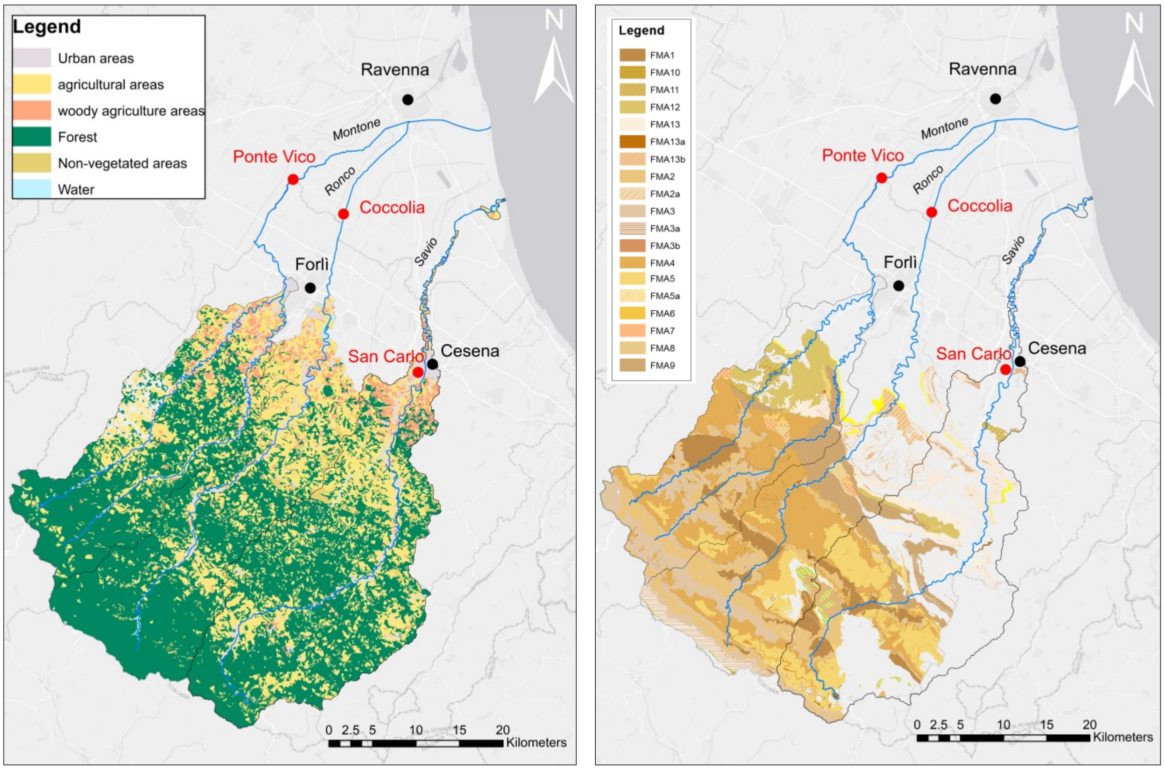


Figure 6.2 - Land use (left) and soil type (right) map, data available on the GeoPortal of Emilia-Romagna Region (http://geoportale.regione.emilia-romagna.it). In the legend of the land use map there is a simplification of the official categories available on the regional map.

The measuring stations are located in the vicinity of the river mouths: in particular the Fiumi Uniti site at a pedestrian bridges while the Savio station at a road bridge at 8 km and 3.5 km upstream of the river outlet, respectively. At both stations water level gauges are present. The cross section at the measuring site is rectangular with a stream bed gradient about 0.0003 m/m for both the Fiumi Uniti and the Savio. The channel width of the Fiumi Uniti cross-section reaches a maximum value of about 60 m in the Fiumi Uniti, the Savio cross-section around 30 m. Bed material of both rivers is sand, with a mean diameter D_{50} of 0.43 mm (Fiumi Uniti) and D_{50} = 0.26 mm (Savio). Tidal excursion at the river mouths is almost restricted (about 0.80-0.90 m in syzygy). Despite this, backwater effects are present with low water discharge conditions but completely irrelevant in case of floods.

6.4 Methods

6.4.1 Water discharge measurements

The flow discharge measurements were made at equally spaced verticals of the active channel on both rivers: five and three verticals were established for the Fiumi Uniti and the Savio, respectively. Flow velocity was measured from the bridge using a standard USGS AA type current meter mounted on a USGS Type E crane. Flow velocity measurements were done following the two point method, i.e. measuring the velocity at 0.2 and 0.8 of the water depth below the surface. Water discharge was calculated by the velocity-area method.

6.4.2 Direct bedload measurements

Bedload measurements were conducted in 2005 and 2006 by previous authors (Billi et al., 2017) then stopped and restarted in 2017 and concluded in 2019, reaching a significant number of data which amount at 23 for the Fiumi Uniti and 14 for the Savio case. Direct bedload measurements were made at fixed verticals standing on the bridge with the use of a Helley-Smith bedload sampler (US BL-84) mounted on a standard USGS Type A crane with four wheels. The sampling verticals used for bedload measurements were the same used for discharge measurements. Moreover, for consistency the same vertical location were used for all measurements. The US BL-84 cable-suspended bedload sampler includes a 29.5 kg solid-steel round-stock bar frame with tails fins; it is equipped with a 46 cm long sample bag with 0.1 mm of mesh connected to the nozzle and the back part of the sampler; a rubber “O” ring attaches the bag to the nozzle, permitting adherence to it (Emmet, 1979). With a square entrance of 7.6 x 7.6 cm, the sampler has an area expansion ratio (ratio of nozzle exit area to entrance area) of 1.4. According to the rate of water changes identified by a staff gauge installed on the bridge, bedload measurements were taken from the bridges at variable time intervals depending on flow conditions. Sampling duration was calibrated at about 10 minutes in case of high flow, while 20 minutes were necessary in condition of low transport. This calibration has been made after few attempts in the field in order to ensure the collection of a sufficient sample volume (Boiten, 2003). Moreover measurement time has been maintained sufficiently long to capture adequately both low and high transport rate, as well as short enough to minimize variation in water discharge (Muhammad, 2019).

To investigate relationship between sediment discharge and grain size, sediment samples collected in field were analyzed in laboratory. In order to perform the grain size distribution, each sample was cleared of alien materials (for example vegetation debris). Then after having dry-sieve and shaking each sample through a standard Rotap (with ½ phi scale), grain size distribution was obtained of each collected samples.

Total bedload mass of each collected sample was weighted and, through calculations which include measuring time and bedload sampler intake geometry, bedload discharge was calculated for each vertical (q_{bi} in kg s^{-1} m^{-1}). Bedload discharge of each flood was then calculated as the sum of bedload discharges of any single vertical. The average bedload transport rate was integrated over the active cross section of the river (which differs from the full wet river cross section) and total sediment transport in a given day was then obtained (Q_{bi} in ton/day), as expressed below:

$$Q_{bi} = \sum_{i,j=1}^{n}(q_{bj} \cdot w_j \cdot 86.4) \quad (6.1)$$

where q_{bj} is the bedload discharge of any single vertical (in kg s^{-1} m^{-1}), w_j is the active cross section of the river (m) and 86.4 is the unit conversion factor (for time and mass).

6.4.3 Bedload transport formulae estimation

Bedload transport formulae have been extensively developed in order to predict bedload transport in rivers. These formulae range from simple regressions to complex multi-parameter formulation with different percentage of success (Barry et al., 2007; Khorram and Ergil, 2010; Zanke, 2001). Among the most well-known formulae available in literature, five of them (Meyer Peter and Muller, 1948; Meyer Peter and Muller modified by Wong & Parker, 2006; Bagnold, 1980; Martin, 2003; Recking, 2010) have been implemented in this study to estimate bedload transport rates against observed data in the studied locations. The models cited were selected because they are renowned, they are based on bed-shear stress concept (Meyer Peter and Muller, 1948; Meyer Peter and Muller modified by Wong & Parker, 2006); or on stream power concept (Bagnold, 1980; Martin, 2003); or taking into consideration the fact that he presence of different bed morphologies affects bedload in a different way (Recking, 2010). The ability of the equation to predict the bedload transport rate in the study sites was made taking into consideration the discrepancy ratio (r) between predicted (Q_{bp}) and measured values (Q_{bm}), (Khorram and Ergil, 2010; López et al., 2014). In order to better understand assessment of performance of the considered bedload formulae, a short deepening follows below.

Meyer Peter and Muller (1948) and revised version of Wong & Parker(2006)

Meyer Peter and Muller (1948) equation is probably the most widely used equation for bedload transport estimation in natural rivers (HEC, 1991;Nicholas, 2000;Church and Hassan, 2005; de Linares, 2007).This formula is based principally on experimental undertaken in the Eidgenössiche Technische Hochschule lab of Zurich, Switzerland with data of Gilbert of 1914 (García and Sala, 1998). The classical equation was express by Chien (1954) in terms of the excess shear stress, as in Equation 6.2.

$$q^* = 8(\tau^* - \tau_{cr}^*)^{\frac{3}{2}} \quad (6.2)$$

Where q^* is the dimensionless bedload rate , τ^* is the dimensionless shear stress and τ_{cr}^* is the critical dimensionless shear stress, equal to 0.047. τ^* is defined by means of Shileds (1936) parameter, i.e. as espressed in Equation 6.3:

$$\tau^* = \frac{\tau}{\sqrt{(s-1)\rho g D}} \quad (6.3)$$

Where $s$is the specific weight of the sediment, g is the gravity acceleration, ρ the sediment density and D the representative sediment diameter (here (D_{50} is used).

The revised MPM (1948) equation was later revised by Wong & Parker in 2006, assuming the following form:

$$q^* = 3.97(\tau^* - \tau_{cr}^*)^{\frac{3}{2}} \quad (6.4)$$

Where the critical dimensionless shear stress was modified from 0.047 to 0.0495.

Wong & Parker (2006) revisiting was done since they evidenced an unnecessary bed roughness correction proposed by MPM (1948) which seemed to be inappropriate for the characterization of mobile bed rough conditions in rivers.

Bagnold (1980) formula

Bagnold (1980) equation is one of the most well known formulae which have been implemented using the stream power. The original expression has the form:

$$q_b = \frac{\gamma_s}{(\gamma_s-\gamma)} q_{b*} \left\{\frac{\omega-\omega_0}{(\omega-\omega_0)_*}\right\}^{\frac{3}{2}} \cdot \left(\frac{Y}{Y_*}\right)^{-\frac{2}{3}} \cdot \left(\frac{D}{D_*}\right)^{-\frac{1}{2}} \quad (6.5)$$

Where: γ_s and γ are the specific gravity of the sediment and of the fluid, respectively (kg m^{-1} s^{-1}); Y is the mean water depth (m); D is the mode size of the bed material (usually denoted by D_{50}); ω is the unit stream power (kg m^{-1} s^{-1}) and ω_0 is the critical stream power expressed as:

$$\omega_0 = 290 D^{\frac{2}{3}} \log(12Y/D) \quad (6.6)$$

Furthermore:

$$q_{b*} = 0.1 \text{ kg m}^{-1} \text{ s}^{-1} \quad (6.7)$$
$$(\omega - \omega_0)_* = 0.5 \text{ kg m}^{-1} \text{ s}^{-1} \quad (6.8)$$
$$Y_* = 0.1 \text{ m} \quad (6.9)$$
$$D_* = 1.1 \cdot 10^{-3} \text{ m} \quad (6.10)$$

Martin equation (2003)

As a simple correlation between stream power and bedload transport rate is defined by Martin in 2003. Based on bedload data found in Gomez and Church (1989), it assumes the following relation:

$$q_b = 0.0505\ \omega^{0.89} \quad (6.11)$$

Where ω is the unit stream power (kg m^{-1} s^{-1}) .
The formula proposed by Martin in 2003 does not include terms as water depth, grain size or threshold of transport.

Recking equation (2010)

Recking equation (2010) is expressed as:

$$q_b = \Phi \cdot \sqrt{g(s-1)D^3} \qquad (\text{m}^3 \text{ m}^{-1} \text{ s}^{-1}) \quad (6.12)$$

Where Φ is the Einstein (1950) parameter equal to:

$$\Phi = 14\ \frac{\tau^{*2.5}}{1+\left(\frac{\tau_m^*}{\tau^*}\right)^{\beta-10}} \quad (6.13)$$

$$\tau_m^* = 0.26\, S^{0.3} \tag{6.14}$$

Where τ^* is the dimensionless shear stress, i.e. equal to $\tau/[(\rho_s - \rho)gD]$; τ is the shear stress equal to ρgRS, ρ is the water density (in kg/m^3), ρ_s is the sediment density (in kg/m^3), g is the gravity acceleration (m/s^2); D is the grain size diameter (D_{84} is used), in m; R is the hydraulic radius (or the water depth) expressed in m; S is the energy gradient slope.
Transformation of q_b from m^3 m^{-1} s^{-1} into kg m^{-1} s^{-1} has been done considering the sediment density ρ_s equal to 2650 kg/m^3 as suggested by Recking et al. (2016).

6.4.4 Hydrodynamic modeling

A quasi steady flow routing model was realized integrating GIS and HEC-RAS approaches to determine the hydrodynamic condition along both the studied rivers. The model has been realized with an overall of 31 (for the Fiumi Uniti) and 20 (for the Savio) surveyed cross sections almost equally spaced (about 300 m between each other) provided by the Italian National Hydrographic Service. Boundary conditions of the model were the observed flood hydrograph (upstream) and the recorded sea level (downstream). Both river models were calibrated using the observed water level for all available flood events (23 for the Fiumi Uniti and 14 for the Savio). Considering actual geometry and the roughness distribution along the wetted perimeter, the representative Manning's coefficient results to be equal to 0.019 m-1/3s s and 0.029 m-1/3s for the Fiumi Uniti and Savio, respectively.

6.4.5 Mid-term reconstruction of bedload transport

6.4.5.1 Water discharge reconstruction

Water discharge reconstruction for a total period of twelve years (2007-2018) has been performed in the two following different ways: (a) using available data of average daily water discharges published in the annual reports (Annali Idrologici) of the Italian Hydrographic Service of the Emilia-Romagna region; (b) using available data of water level accessible on the online platform (Dext3r) of the regional prevention, environment and energy agency of the Emilia-Romagna Region. In the first case (a), available daily water discharges have been used to rebuild the changes in flow rate during each hydrographic year. The approach has been applied considering established rivers stations located at the closure of the basins, and in particular they are: Ponte Vico (Montone); Coccolia (Ronco) and San Carlo (Savio), (Figure 6.2). At these locations radar sensors for hydro-meteorological monitoring of the Italian Hydrographic Service of the region are collocated. Confirming that the Fiumi Uniti derives from the unification of the Montone and Ronco basins, which both hydraulically behave in equivalent way, it has been possible to assume that the water discharge of the Fiumi Uniti derives from the sum of the two water discharges. Instead the second approach (b), which concerns the same established rivers stations of the first method (Figure 6.2), involves available data of water level with half-hourly durations. The approach consists in the application of the yearly rating curves (Annali Idrologici) to transform water level in water discharges, obtaining in this way the complete dataset of flow rate during each hydrographic year.

6.4.5.2 Mid-term bedload discharge reconstruction based on semi-hourly data

Bedload discharge reconstruction has been done considering that the threshold condition of sediment motion is known for both monitored rivers. The critical discharge has been observed to be approximately equal to 17 m^3/s and to 19 m^3/s for the Fiumi Uniti and the

Savio river respectively (Cilli et al., 2018). Considering these threshold values, it has been possible to exclude from the complete dataset of flow rate of each hydrographic year all water discharges lower than these values. Correlation between water discharge and bedload discharge obtained in the field on both rivers (see Chapter 6.4.2 and 6.5.2) has been successively applied to the new cleaned up dataset, resulting yearly bedload discharges reconstruction for the total period which goes from 2007 to 2018. In order to calculate bedload discharge over the whole effective channel cross section, it has been applied a prefixed active and effective channel width equal to 40 m and to 18 m to the Fiumi Uniti and Savio calculations, respectively.

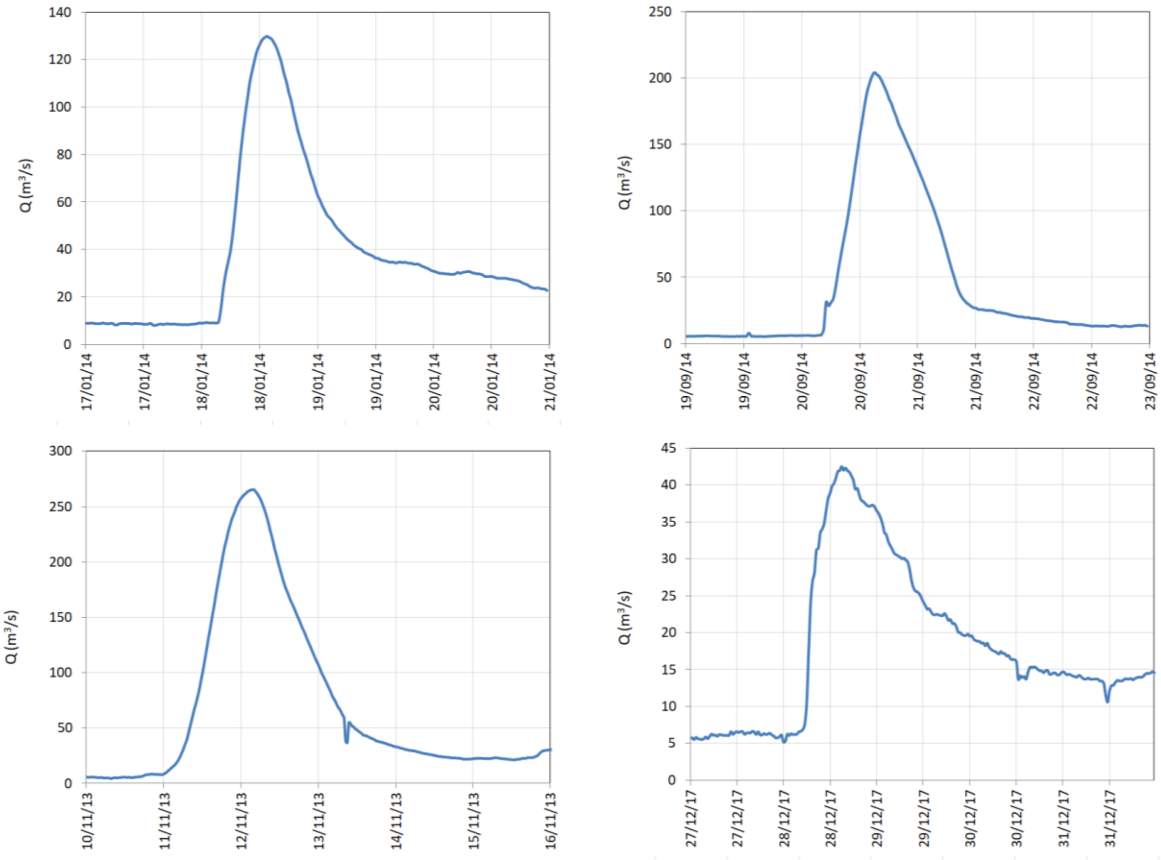
6.4.5.3 Deepening on difference between average daily water discharges and real water discharges hydrograph and its effect on sediment transport calculation

In order to perform a correct mid-term reconstruction of bedload transport, previously a proper preliminary hydrological analysis has been done focusing on the difference which exists between daily water discharges and real water discharges hydrograph. It was decided to make this deepening only on the Fiumi Uniti since it was believed that this river could have greater complications than the Savio. These complications are essentially referred to the fact that the Fiumi Uniti derives from the unification of two rivers (Montone and Ronco), which unifying in a single stream, may show difference in the synchrony of the transfers of the water flows from upstream to downstream.

Discrepancies between these two values (daily water discharges and real water discharges) have been investigated resulting in a methods which can be applied to both studied rivers. Regarding the last approach, a wide dataset of twelve years of floods (2007-2018) has been analyzed for an established river station on each river channel (stations highlighted in red, Figure 6.2). Observed water level dataset obtained at the river stations have been transformed in water discharge dataset thanks to rating curves available on the reports (Annali Idrologici) of the Italian Hydrographic Service of the region (see Chapter 6.4.5.1). The obtained water discharge dataset has permitted to examine the hydrograph shape of the most significant flood events, identifying some parameters which have permitted to establish a common computational procedure applicable to all basins, which is explained below.

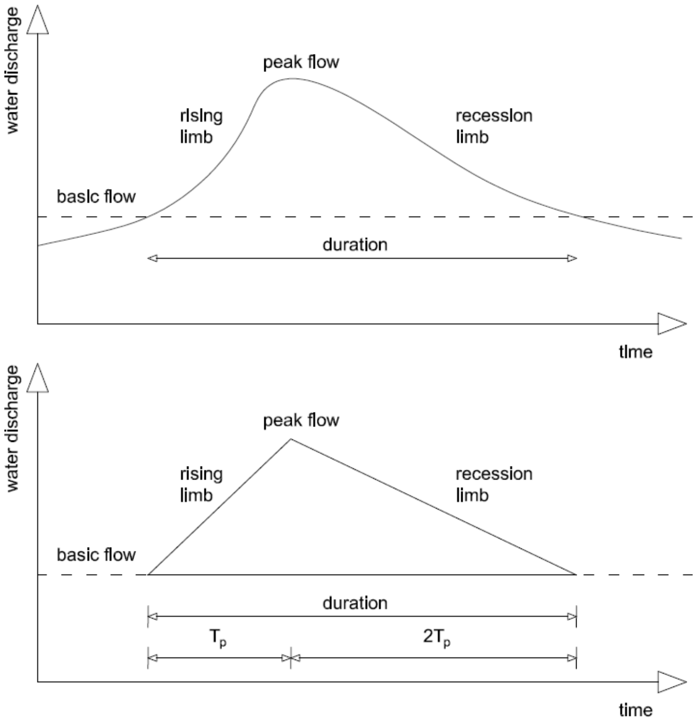
Considering that the most frequently used catchment response time parameters are the time of concentration (T_c), lag time (T_l) and time to peak (T_p), which are normally defined in terms of the physical catchment characteristics and/or distribution of effective rainfall and direct runoff (USDA NRCS, 2010), the presented method involves first of all the concept of time of concentration of the studied river basins. With time of concentration $\mathrm{T_c}$ of a general river basin is meant the time from the start of the total runoff to the time of the peak discharge of the total runoff over the river basin (McCuen, 2009). With lag time T_l instead is generally defined the time between the centroid of effective rainfall and the peak discharge of the resultant direct runoff hydrograph. Peak time T_P, which is used in many hydrological applications, can be defined as the time from the start of effective rainfall to the peak discharge in a single-peaked hydrograph (USDA SCS, 1985). Among these, the one of greatest interest in order to reconstruct the actual hydrogram from the average daily flow values recorded in the observation period is the peak time T_P.

Nineteen "ordinary" full events have been identified during the twelve-year (2007-2018) monitoring period. The procedure basically consists of identifying the hydrograph shape of the occurred floods which, for simplification, has been approximated to a triangle with linear rise and recession. The analysis of the hydrographs shows a basin's response to rain precipitations characterized by the following proportion: a growth time (equal to the time T_P) and two emptying times (i.e. $2T_P$), as shown in Figure 6.3.



6.3 – Example of some observed hydrograph where the growth time is equal to the time T_P and the emptying times equal to $2T_P$.

The shape of the observed hydrograph of floods also suggests the possibility of schematizing them with a triangular wave shape. The schematic representation is shown in Figure 6.4.



6.4– Simplification of flood hydrograph to a triangular wave shape.

Having assumed the total duration equal to $3T_p$ of any triangular hydrograph, the important element which need to be modeled is the correlation between peak flow and the average daily water discharge.

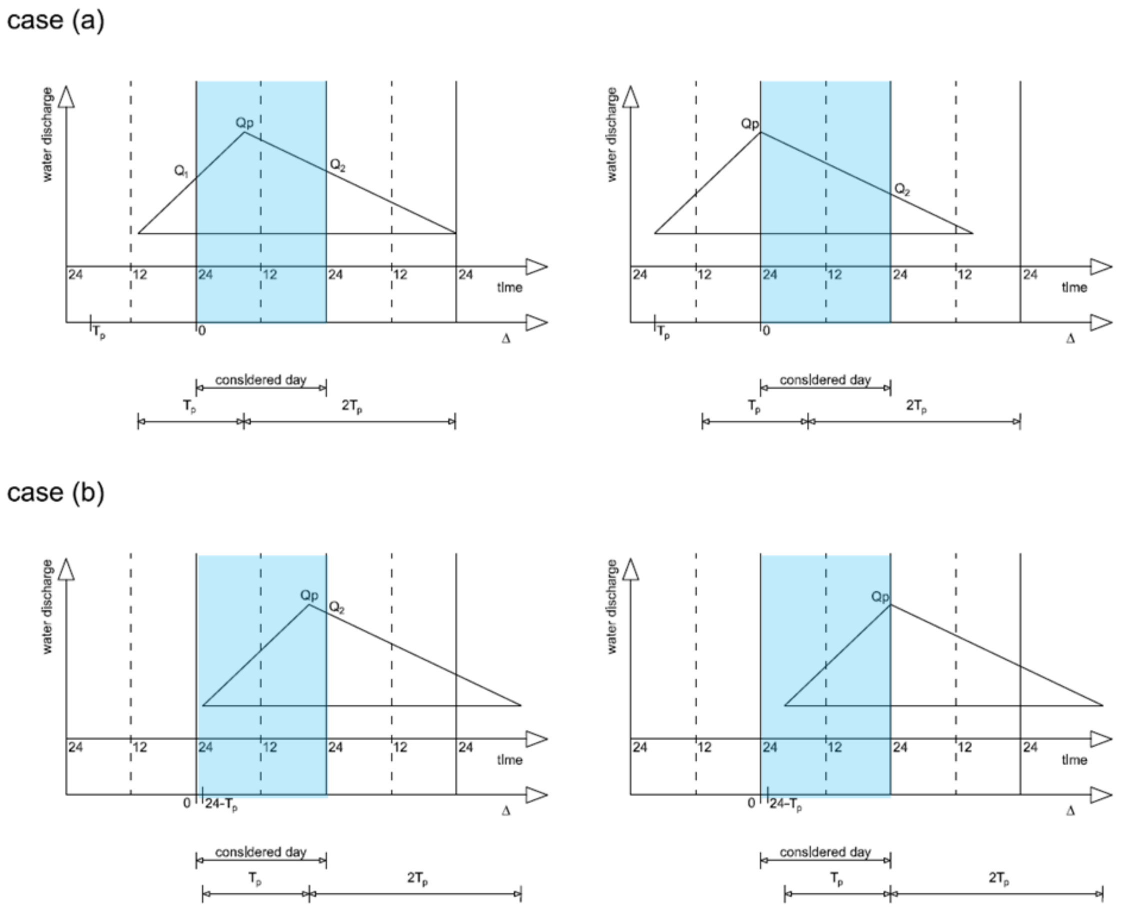
Considering that the great majority of the significant floods show peak times between 8 and 24 hours (8 <T_p<24 h), the analysis was focused on these cases. On this purpose, two main cases have been developed: (a) peak flow of the flood occurs in the established day with beginning of the hydrograph the day before the peak; (b) peak flow of the flood occurs in the established day with beginning of the hydrograph during the same day (Figure 6.5). When the peak of the hydrograph coincides with the beginning or the end of the considered day, the two cases are considered to be equivalents. Considering the cases just described, they are both limited to situation in which $8h < T_p < 24h$. Moreover, concerning this assumption, an additional parameter has been included in the model: i.e. the duration of the flood on the day before the peak flow, also called Δ. Considering both described scenarios, this parameter can range between $-T_p$ and $24 - T_p$ (Figure 6.5). Taking into account the herein described model and the parameters involved it is possible to define correlations between discharge peak of the hydrograph Q_p and daily water discharge $\overline{Q_d}$. For the case (a) the expression is showed in Equation 6.15, while for case (b) in Equation 6.16.

$$Q_p = \overline{Q_d} \cdot \frac{96T_p}{2{T_p}^2 - 2\Delta^2 + (24 - T_p + \Delta)(5T_p - 24 - \Delta)} \tag{6.15}$$

$$Q_p = \overline{Q_d} \cdot \frac{96T_p}{(24 - \Delta) \cdot 2T_p + (3T_p - 24 + \Delta) \cdot (24 - T_p - \Delta)} \tag{6.16}$$

The analysis was extended to all significant floods (Q_p > 27m^3/s) of the 2007-2018 registration period and the results are illustrated and commented on in the following paragraph (6.5). The aforementioned analysis is valid for the ordinary floods regime;

instead in the field of extraordinary floods (flows of annual duration of the order of $3T_p$) an additional analysis related to the extreme values is necessary.



6.5 – Simplification of flood hydrograph for correlation between peak flow and the average daily water discharge. Case (a) represente the case in which peak flow of the flood occurs in the established day with beginning of the hydrograph the day before the peak; case (b) represent the case in which peak flow of the flood occurs in the established day with beginning of the hydrograph during the same day. Extreme condition of each case is represented on the right part of the figure, resulting to be the same condition. In light blue it is highlighted the considered day, i.e the day in which the peak should occurs.

As already anticipated in the paragraph 6.2, a statistical analysis of outflows and projection in the medium to mid-term prospects of the annual duration curve of the average daily flows has to be investigated. Here the estimation of the concentration time T_c is reported, since it is also useful in terms of comparison with the values of the peak times T_p previously analyzed over the 2007-2018 time series. The time of concentration T_c (definition reported in paragraph 6.4.5.2) of all river basins (Montone, Ronco, Savio) has been estimated using the following indirect methodologies: (a) estimation through empirical formula of Giandotti (1934); (b) estimation with Soil Conservation Service Curve Number (SCS-CN) methodology (USDA 1986, Chow et al. 1988); (c) estimation based on the observed hydrographs. All approaches have been performed considering that the closure of the basin is Forlì both for the Montone and Ronco rivers, while Cesena for the Savio (Figure 6.1, 6.2). Concerning the first approach, Giandotti's formula (1934) is one of the most extensively used in Italy (Grimaldi, 2012). Here, such formula is here reported:

$$T_c = \frac{4\sqrt{A}+1.5L}{0.8\sqrt{H}} \tag{6.17}$$

Where T_c is the time of concentration (h), A is the river basin (km^2), L is the total length of the main channel (km) and H is the difference between the maximum basin elevation and the outlet elevation (m).
By use of soil type and land use information given by the River Basin Authority, Soil Conservation Service Curve Number (SCS-CN) methodology has been applied (Eq.6.16). The combination of soil type permeability (low, medium and high) and land use (agriculture, pasture, forest and urban areas) information related to the whole watersheds of the considered rivers have permitted to establish first CN parameter and later T_c, as expressed by Equation 6.18.

$$T_c = 0.00227 \cdot L \cdot \left(\frac{1000}{CN} - 9\right)^{0.7} \cdot i^{-0.5} \tag{6.18}$$

6.5 Results

6.5.1 Hydrological condition

Measurements carried out in 2005-06 and 2017-19 have permitted to reach a significant dataset which amount at 23 (for the Fiumi Uniti) and 14 (for the Savio) monitored floods. Already 11 floods were measured by previous authors (Billi et al., 2017) for the Fiumi Uniti river. While the dataset of the Savio river is completely new. In Figure 6.5 are represented the rating curves of the Fiumi Uniti (a) and Savio (b), respectively. The curves follow the mathematical form proposed by Herschy (1985), i.e. following the next equation:

$$q = a(h - h_0)^b \tag{6.19}$$

Where a, b, h_0 represent the coefficients which have to be estimated.

The range of flow discharges goes from 17.27 m^3/s (measured on the 11/10/2005) to 358.16 m^3/s (occurred on the 12/04/2005) for the Fiumi Uniti river.; instead for the Savio the weakest flood measured is of 7.04 m^3/s (on the 07/11/2017) and the highest one of 234.30 m^3/s (on the 15/05/2019). The highest flood measured in this study can be considered the largest floods recorded in the last decade for both study cases. Flow velocity range between 0.2 and 1.66 m/s for the Fiumi Uniti case, while between 0.2 and 1.88 for the Savio river. Instead water depth goes between 1.31 and 4.72 m and from 1 and 5.96 m for the Fiumi Uniti and the Savio, respectively.

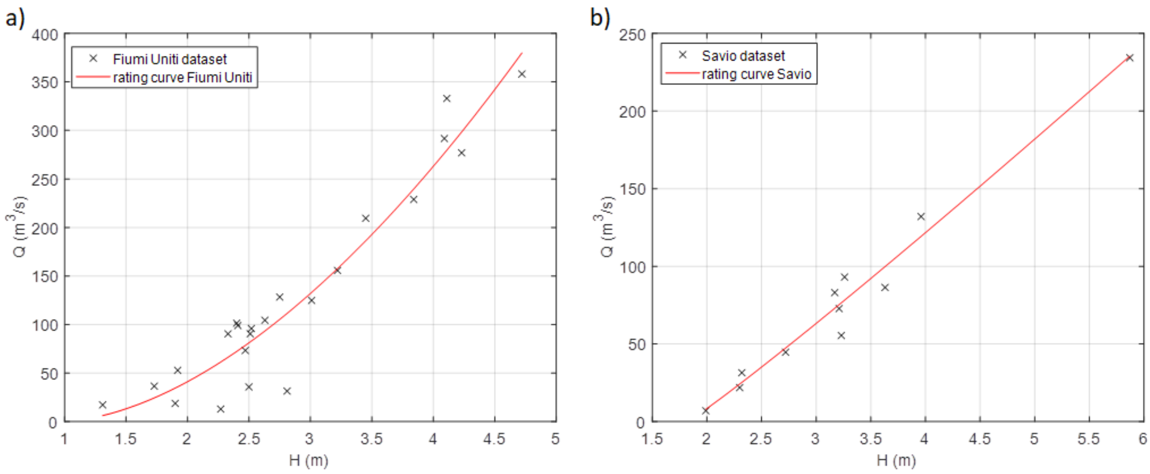


Figure 6.6 – Rating curves and summary of measured data of the Fiumi Uniti River (a) and Savio river (b). Correlation between water discharge (Q) and water depth (H). For the Fiumi Uniti river the equation has the form $Q = 38.68 (H-0.96)^{1.73}$ with R^2 equal to 0.92; the Savio river instead: $Q = 53.15(H-1.82)^{1.06}$ with R^2 equal to 0.97.

6.5.2 Bedload transport and sediment characteristics

Bedload discharge Q_b was also measured during the same monitoring flood events. However in some cases it has not been possible to measure bedload transport because absent or considered negligible. Total amount of Q_b measured consist of 19 (out of 24) events for the Fiumi Uniti and 9 (out of 11) events for the Savio. The Q_b of the Fiumi Uniti ranges between 0.0003 and 0.1460 kg/ms while between 0.0004 and 0.0286 kg/ms for the Savio. Figure 6.6 shows the correlation between flow discharge Q (m^3/s) and solid discharge Q_b (kg/ms) for the studied rivers. In both cases, field observations indicate that bedload was active across the entire cross-section only for big flow discharges, whereas for smaller floods, only the active cross section (i.e. the central portion of the river bed) was involved. Figure 6.6 shows the best fitting follows a power law function where for the Fiumi Uniti the R^2 is equal to 0.63, while for the Savio equal to 0.61. Appreciable bedload discharge associated with smallest flood has been investigated resulting that the weakest flood of 17.27 m^3/s of the Fiumi Uniti and the flood of 19.07 m^3/s can be considered as the critical flow condition for bedload entrainment in the studied reaches. While the threshold value of the Fiumi Uniti (17.27 m^3/s) have already been investigated (Billi et al., 2017), the threshold value of the Savio river was completely unknown. Threshold flows correspond to a critical shear stress τ_{cr} equal to 1.73 N/m^3 (Fiumi Uniti) and to 3.60 N/m^3 (Savio).

Grain size of the transported material highlight the presence of medium to coarse sand, as evidenced in Table 6.1. Sediment transported by the Fiumi Uniti highlight an average D_{50} of 0.50 mm while the Savio of 0.49 mm. Also average D_{90} has been investigated: 1.04 mm is the average D_{90} for the Fiumi Uniti, while 1 mm for the Savio.

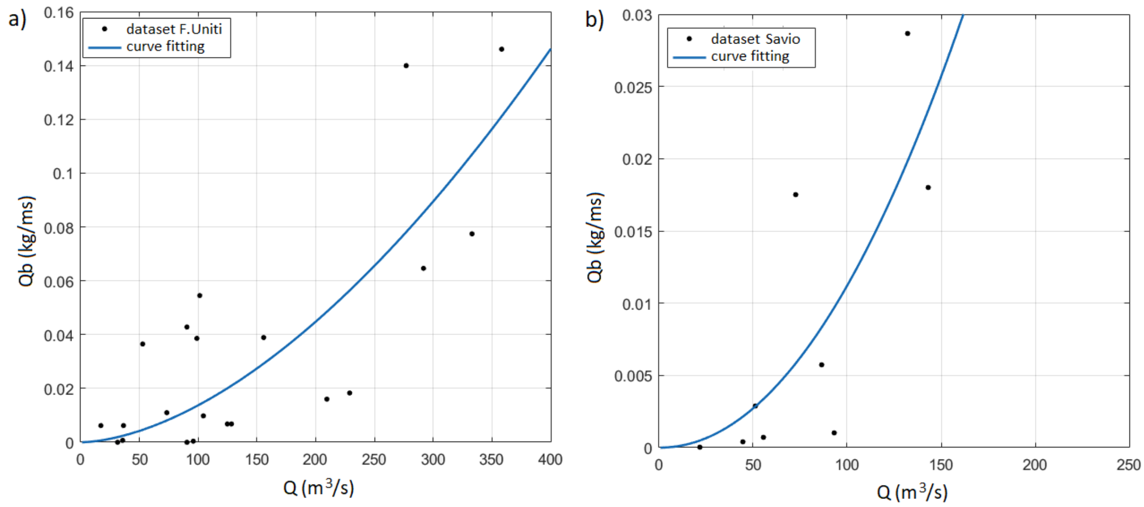


Figure 6.7 – Correlation between flow discharge (Q) and sediment discharge (Q_b) expressed with power law function for the Fiumi Uniti with equation Q_b= 5.30e-06 $Q^{1.71}$ with R-square equal to 0.63 (a) and Savio river with equation Q_b= 9.05e-07 $Q^{2.046}$ with R-square equal to 0.61(b).

Table 6.1 – Sediment size of the transported material for the Fiumi Uniti and Savio rivers.

	D_{50}		D_{90}	
	min. value (mm)	max. value (mm)	min. value (mm)	max. value (mm)
Fiumi Uniti	0.39	0.66	0.63	2.10
Savio	0.41	0.64	0.66	1.80

6.5.3 Comparison of measured and predicted bedload transport

The bedload transport rates were computed using selected predictive methods (i.e. Meyer Peter and Muller, 1948; Meyer Peter and Muller modified by Wong & Parker, 2006; Bagnold, 1980; Martin, 2003; Recking, 2010). The selected equations were tested comparing measured bedload transport values (Q_{bm}) with predicted ones (Q_{bp}). Results obtained are shown in Table 6.2, where there is the comparison of the predictive performance of the five selected predictive methods. Figure 6.8 shows a comparison between all tested methods, indicating Martin (2003) and Bagnold (1980) as methods which seem to provide most reasonable predictions. On this purpose Figure 6.9 highlights discrepancies between the two methods in terms of correlation between water discharge Q and bedload discharge Q_b.

Table 6.2– Comparison of the selected predictive methods. Discrepancy ratio (r) and range of discrepancy is given.

	Fiumi Uniti		Savio	
	r = Q_{bm}/Q_{bp}		r = Q_{bm}/Q_{bp}	
	Mean r	Range	Mean r	Range
Meyer Peter and Muller (1948)	10.12	0-85.71	367.32	37.01-748.28
Meyer Peter and Muller (1948) modified by Wong & Parker (2006)	4.68	0-40.35	103.82	5.76-323.87
Bagnold (1980)	0.07	0-0.54	2.37	0.6.99
Martin (2003)	1.72	0.18-19.13	20.95	2.36-52.73
Recking (2010)	3.96	0-23.90	435.81	21.44-1274.87

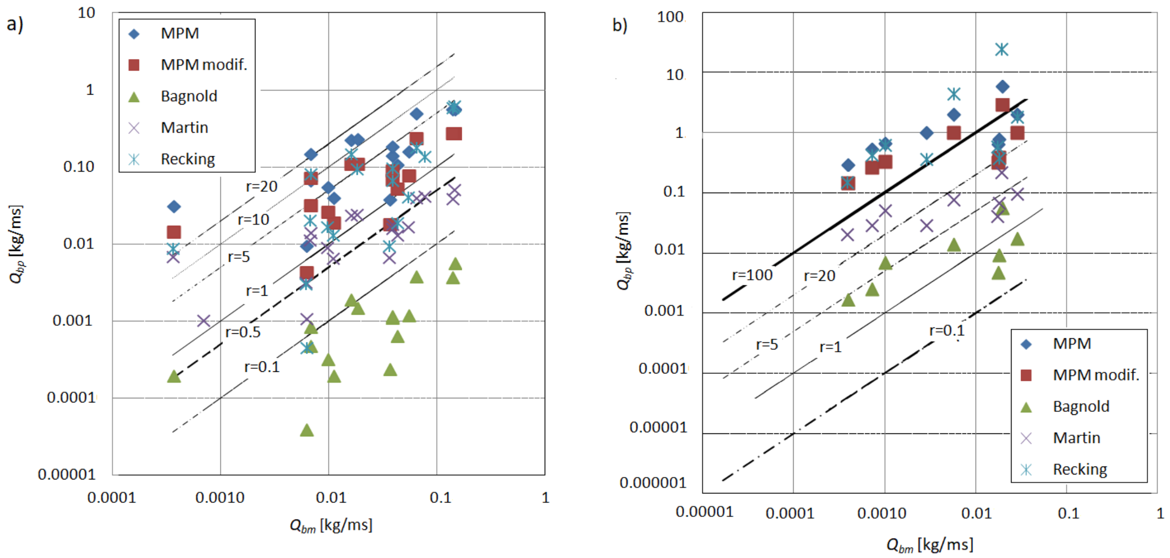


Figure 6.8 – Measured and predicted bedload transport in the Fiumi Uniti (a) and Savio river (b). Plotted lines are referred to the discrepancy ratio (r).

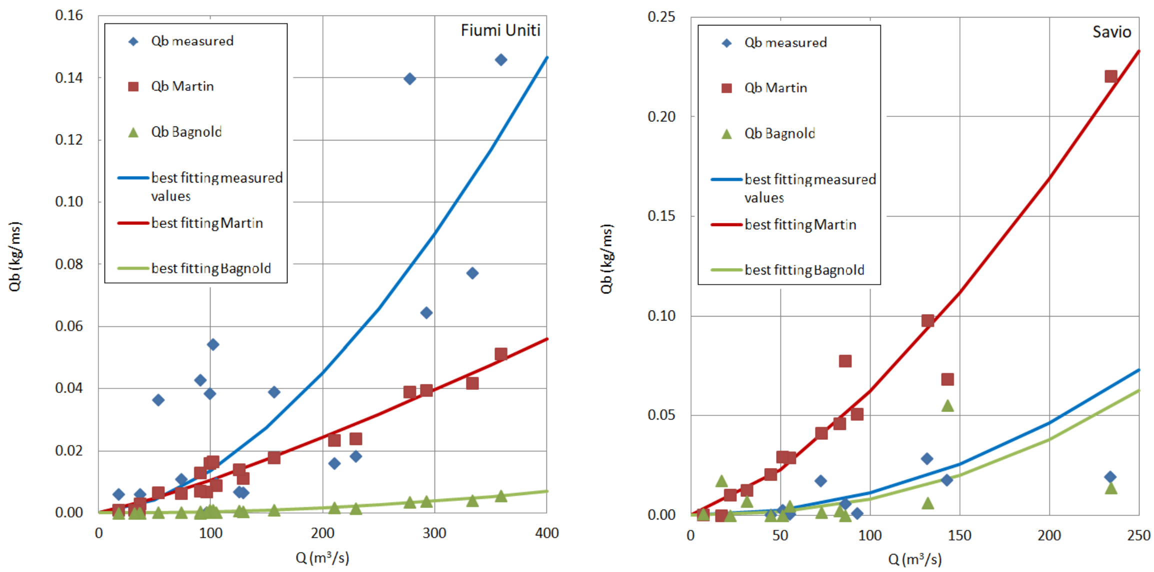


Figure 6.9 – Comparison between measured and predicted bedload transport obtained with Martin (2003) and Bagnold (1980) criteria in the Fiumi Uniti (left) and Savio river (right) .

6.5.4 Mid-term bedload discharge reconstruction

As previously explained in Chapter 6.4.5.2 the mid-term reconstruction of bedload transport on the Fiumi Uniti and Savio river has been performed in two different ways: (a) using available data of average daily water discharges; (b) using available data of water level with half-hourly durations. For simplicity from now on we will call with Q_{ba} the bedload discharge reconstruction obtained with the first method and Q_{bb} bedload discharge reconstruction obtained with the second one. Focusing on the second approach, rating curves of the Montone river at Ponte Vico station, Ronco river at Coccolia station and Savio river at San Carlo station are represented below for the complete period of the analysis (2007-2018), (Figure 6.10).

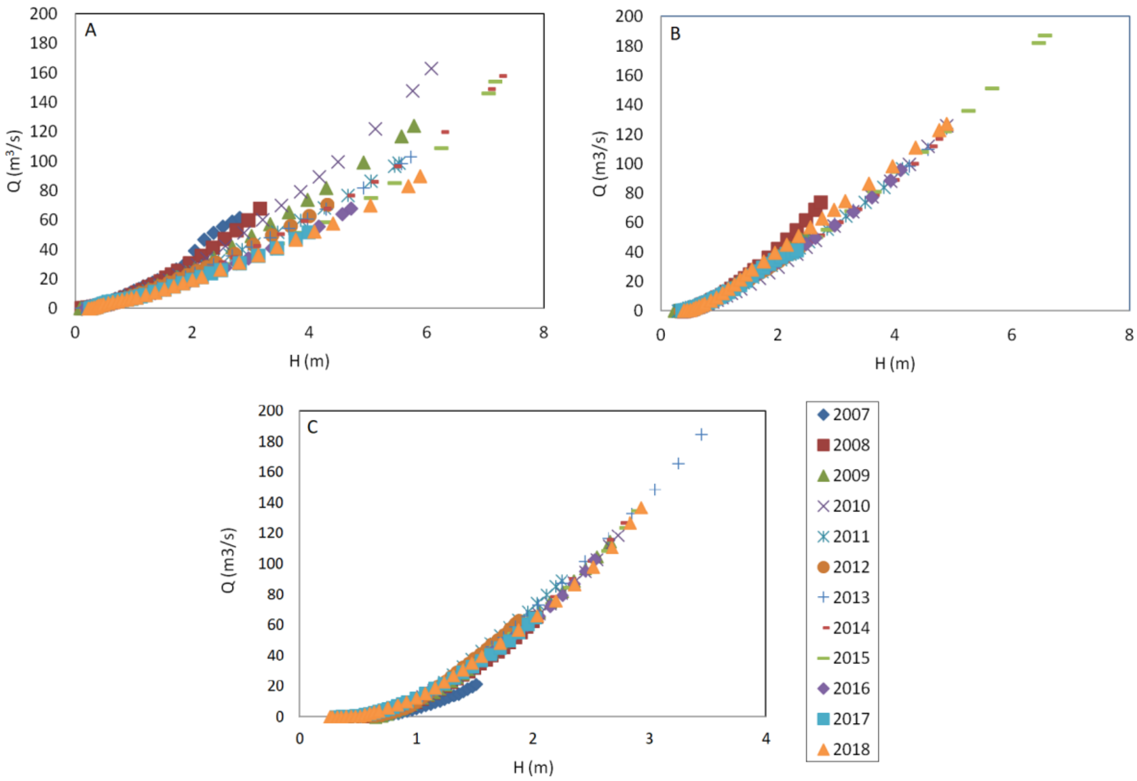


Figure 6.10– Summary of the rating curve of the Montone at Ponte Vico station (A) , the Ronco river at Coccolia station (B) and Savio river at San Carlo station (C) available on the reports of the of the Italian Hydrographic Service of the region. The figure shows the rating curve of each analyzed year used for the water discharge reconstruction of the Fiumi Uniti and Savio river (method b). All rating curve represented express correlation between water discharge Q and water depth H following the power law function.

In Figure 6.11 an example of the reconstruction of water and bedload discharge of the Fiumi Uniti river over the complete mid-term period of twelve years is represented. Reconstruction of water discharges have been performed taking into account that in some cases it has been necessary to clarify the signal of the variable (water level). Abnormal values noted on the data plot have been removed manually without altering the quality of the data. Moreover some years analyzed shows a partially complete dataset. These gaps can be attributed to a loss of signal in the radar sensor of hydro-meteorological monitoring. In these case the calculations have been performed considering only the available data.

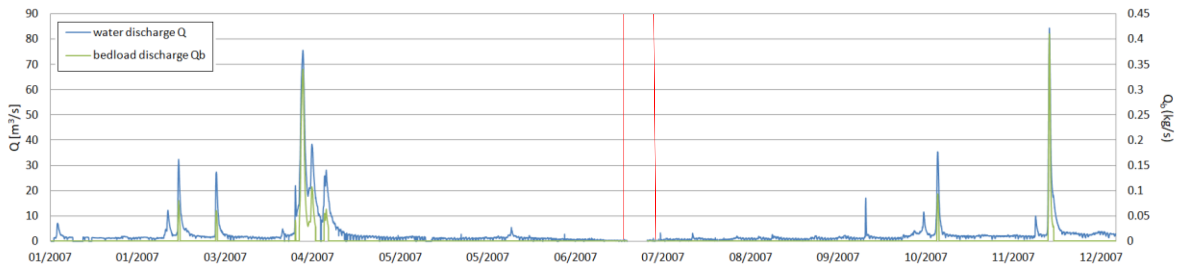


Figure 6.11 – A selected example of water discharge and bedload transport reconstruction of the Fiumi Uniti river on the year 2007. In July, highlighted in red, a period with no data is visible.

Following a comparison of the result obtained with method (a) and (b) is given below in figures 6.12 for the Fiumi Uniti and Savio, respectively.

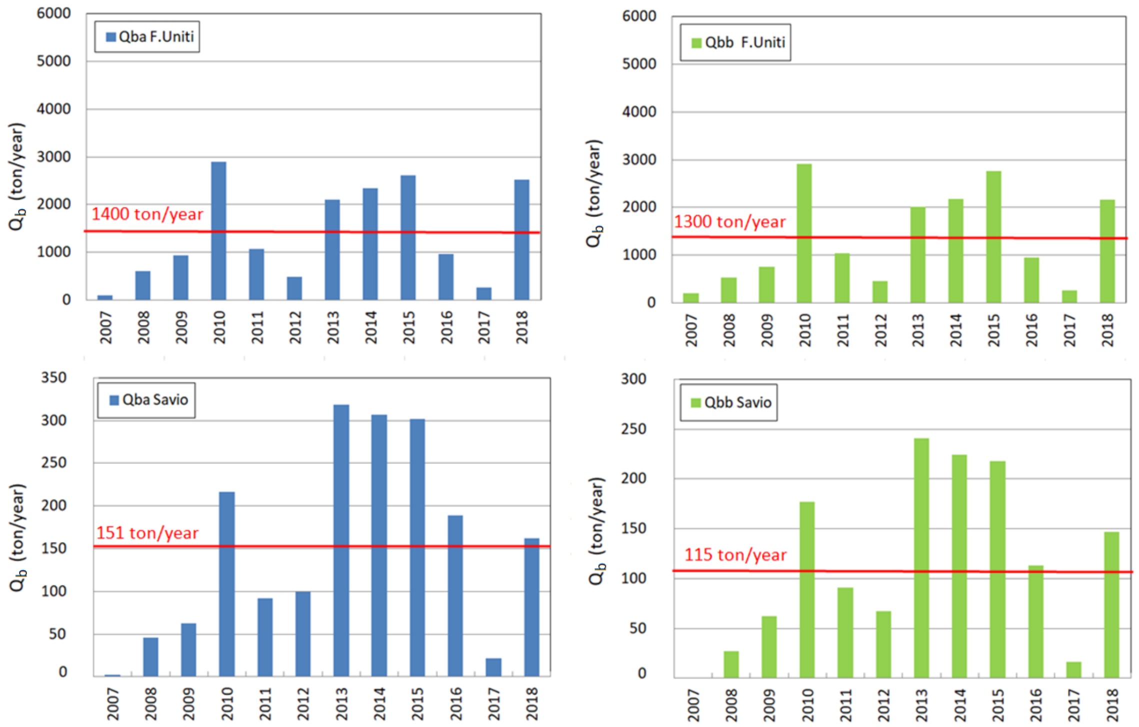


Figure 6.12 – Mid-term bedload discharge reconstruction of the Fiumi Uniti and Savio river with method (a) on the left and (b) on the right. The red line indicates the mean bedload discharge.

Additional comparison of the two methods (a) and (b) has been done considering also the possibility to apply Martin criterion (2003) to the water discharges reconstructed with method (b), i.e. using available data of water level with half-hourly durations. This last approach will be called method (c) for simplicity. Results of Martin criterion (2003) application have been obtained performing the entire water discharge dataset with Hec-Ras program through the use of the hydraulic model previously realized; stream power parameter values obtained from the model have later been applied to Martin (2003) criterion for the whole dataset obtaining results shown in Figure 6.13. Values of mean bedload discharge obtained with Martin (2003) criterion application are 3500 and 970 ton/year for the Fiumi Uniti and the Savio river, respectively. The resulting comparison between the three methods (a,b,c,) is then proposed in Figure 6.13 for both rivers. In Table 6.3, 6.4 are reported the significant values related to the mid-term bedload discharge reconstruction realized with the three methods explained for the Fiumi Uniti and Savio river, respectively. In Table 6.5 cumulative values of the Fiumi Uniti and Savio river are reported. Transformation to cubic meter has been performed considering that the density of the sediment ρ_s is equal to 1600 kg/m^3.

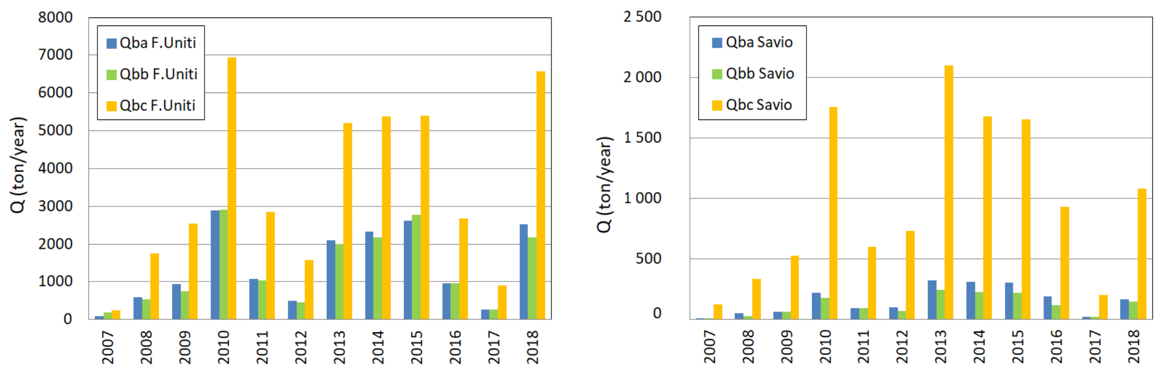


Figure 6.13 – Comparison of mid-term bedload discharge reconstruction of the Fiumi Uniti and Savio river with methods a, b and c (i.e. with the application of Martin criterion).

Table 6.3– Comparison of the significant values related to the lmid-term bedload discharge reconstruction realized with the three methods explained for the Fiumi Uniti river.

	Fiumi Uniti					
	$Q_b(a)$		$Q_b(b)$		$Q_b(c)$	
	ton/year	*m³/year*	*ton/year*	*m³/year*	*ton/year*	*m³/year*
min	95	59	195	122	246	154
max	2898	1811	2912	1820	6945	4341
mean	1406	878	1350	843	3502	2189

Table 6.4– Comparison of the significant values related to the mid- term bedload discharge reconstruction realized with the three methods explained for the Fiumi Uniti river.

	Savio					
	$Q_b(a)$		$Q_b(b)$		$Q_b(c)$	
	ton/year	*m³/year*	*ton/year*	*m³/year*	*ton/year*	*m³/year*
min	2	1.37	1	0.46	118	74
max	318	199	241	150	2098	1311
mean	152	95	115	72	974	609

Table 6.5– Cumulative values of the Fiumi Uniti and Savio river.

		Fiumi Uniti	***Savio***
$Q_b(a)$	ton	16867	1819
	m³	10542	1137
$Q_b(b)$	ton	16195	1383
	m³	10122	865
$Q_b(c)$	ton	42022	11688
	m³	26264	7305

Furthermore considering the Fiumi Uniti river a deepening on time of concentration T_c has been performed. Results obtained shows that T_c estimated with Giandotti's formula (1934) are equal 14 and 17 hours for the Montone and Ronco river, respectively. Instead the application of the Soil Conservation Service Curve Number (SCS-CN) methodology have resulted in 20 and 23 hours for the Montone and Ronco river, respectively. Finally, considering the entire dataset analyzed on the Fiumi Uniti with nineteen hydrograph of the most significant flood events has resulted in a variation between 10 and 30 hours. Obtained results of this last detailed investigation are reported in Figure 6.14, where time of peak T_p, parameter Δ, peak of flow discharge Q_p and average daily water discharge $\overline{Q_d}$ are reported. Figure 6.15 expresses the relation found between time of concentration T_c and the peak of flow discharge Q_p. In addition computations of theoretical and empirical rations between $\overline{Q_d}$ and Q_p are presented below (Figure 6.13). With theoretical computations is meant ratio $\overline{Q_d\ /\ Q_p}$ calculated through the use of Equation 6.17 and 6.18; while for empirical ratio is meant the mathematical ratio between $\overline{Q_d}$ and Q_p.

Data	T_p	Δ	$\overline{Q_d}$	Q_p
09/12/2007	18	12	73.80	84.47
26/02/2007	13	0	18.62	27.49
12/04/2008	12	4	31.40	73.03
13/01/2008	12	7	65.30	107.50
23/12/2009	25	8	176.70	339.08
04/03/2010	24	0	289.00	364.87
16/12/2012	12	6	44.60	91.92
19/03/2013	20	14	189.10	298.27
12/11/2013	27	24	167.50	265.30
20/09/2014	11	0	124.20	203.91
18/01/2014	10	0	61.30	129.70
05/03/2014	30	14	98.60	355.10
30/07/2014	12	0	26.80	82.63
20/09/2014	11	0	124.20	203.90
04/12/2014	18	13	90.10	209.00
02/05/2016	20	0	43.66	138.50
28/12/2017	10	0	36.40	42.51
25/02/2017	17	0	54.20	82.65
02/02/2018	15	0	45.80	166.50

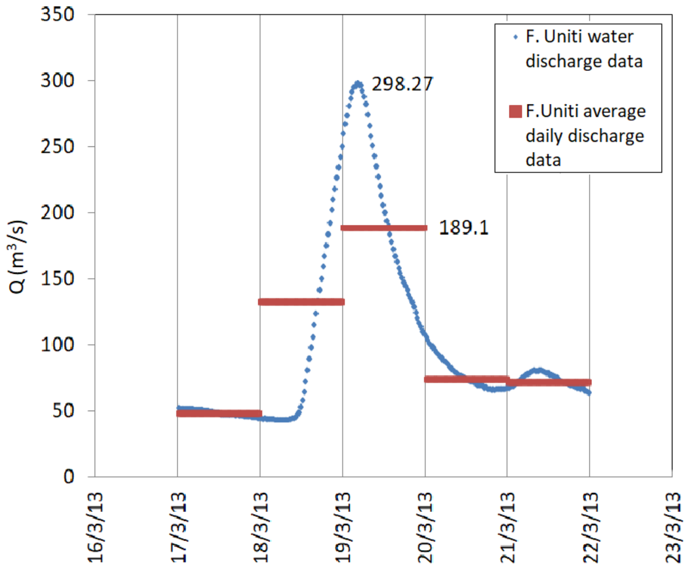


Figure 6.14 – Parameters of the most significant flood events occurred on the Fiumi Uniti river. An example of one of the flood hydrographs is given, highlighting discrepancies between water discharge data and average daily discharge ones. From this example is possible to see the parameters as total flood duration, time of concentration and duration of the flood on the day before the peak flow, Δ.

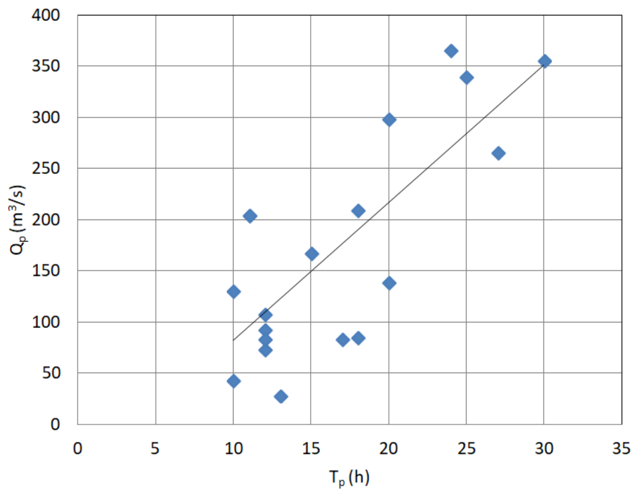


Figure 6.15 – Correlation between time of peak T_P and peak of flow discharge Q_p on the Fiumi Uniti river. The linear regression showed has the following equation: Q_p = 13.5 T_c-52.9. The R^2 is equal to 0.59.

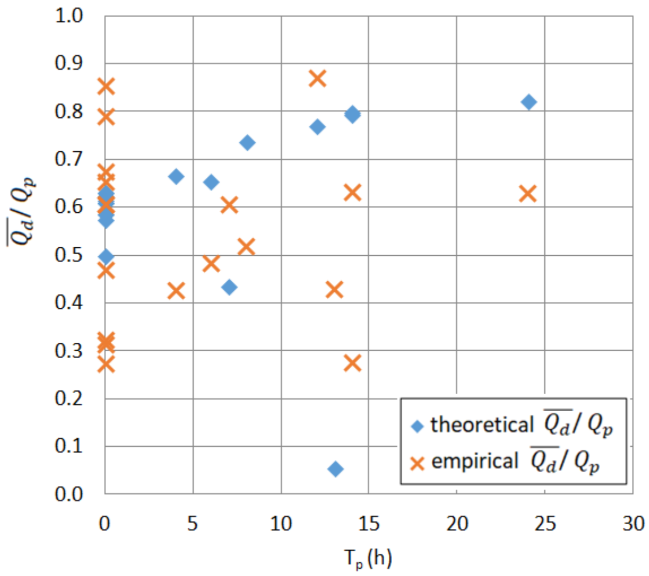


Figure 6.13 – Correlation between time of peak T_P and parameter $\overline{Q_d}\ /\ Q_p$ of the most significant flood events occurred on the Fiumi Uniti river (period 2007-2018).

6.6 Discussion

Measurements carried out on the Fiumi Uniti and Savio rivers have permitted to create rating curves which were inexistent until the present study. Rating curve obtained on the Fiumi Uniti shows a dataset wide enough to cover an ample range of liquid flow rates, ranging from 13 to 358 m^3/s, with a total number of observation of about 24 floods. Correlation between water discharge and water depth obtained for the Fiumi Uniti follows the Herschy (1985) expression of $Q= 38.68\ (H-0.96)^{1.73}$. Observations demonstrate that, with the exception for three spot data where there is high water level (between 2.25 and 2.75 m) with low water discharge (lower than 40 m^3/s), the point cloud of the Fiumi Uniti seems to be well consolidated. Comparing the actual dataset with the one already available in literature (Billi et al., 2017), it is possible to affirm that is has been increased, adding in particular extra significant points in the range of medium-high flow which previously were missed. Concerning the Savio river, the dataset obtained is quite accurate in terms of flow discharges lower than 150 m^3/s. Water discharges data between 150 and 250 m^3/s are unfortunately missed. Time condition is one of the most important factors which make measurements so difficult. On this purpose, it is possible that some flood have occurred at night making measurements impossible to sample. Hence the lack of these data. The Savio rating curve shows an high point of water discharge at about 230 m^3/s which can be considered the largest floods recorded in the three years of monitoring as well as presumed one of the most remarkable flood recorded in the last 10 years. Despite the gap on flow discharges between 150 and 250 m^3/s, the detected points of the Savio rating curve return reasonable hydrometric conditions. On this purpose it is logically expected that, in presence of further data, the behavior of the rating curve stays the same. The Savio rating curve seems to represent in a good way the missing range, therefore it is possible to affirm that also the point cloud of the Savio river seems to be well consolidated. Further measurements are necessary to confirm this hypothesis.

Direct bedload measurements carried out at the measuring site have shown the existence of bedload transport which can be considered quite appreciable. Bedload contribution to the sea has prevalently shown sediment size which is between medium to coarse sand in both cases. Grain size of the transported material has an average D_{50} of 0.50 mm and of 0.49 mm for the Fiumi Uniti and the Savio case, respectively. Correlation between flow discharge (Q) and sediment discharge (Q_b) on both rivers have shown coefficient of determination, R^2, equal to 0.63 and 0.61 on the Fiumi Uniti and Savio, respectively. Power law function for both rivers is the best fitting correlation found. Concerning the Fiumi Uniti it is worth noticing the presence of several value of low transport associated with high water discharges values. This is evident for water discharges around 100 m^3/s and 200 m^3/s. It is important to consider the presence of an high scatter related to value of water discharge equal to 100 m^3/s. The results of the investigations show that during the highlighted floods, bedload transport did not occur in the complete active portion of the channel but only in some of its subsections, resulting in a difference in the total computation of bedload transport. Also in the case of water discharge around 200 m^3/s, evidences in different amount of sediments sampled on the central and lateral sides of the active portion of the channel explain the difference in the bedload discharge performed. Despite these few cases, this evidence is not a rule. Differently from the Fiumi Uniti, the Savio river seems to perform lower bedload transport rates even in case of high water discharges. The Fiumi Uniti shows appreciable bedload transport for values of water discharges higher than 100 m^3/s; while the Savio for values higher than 50 m^3/s. Unfortunately only a few points exceed this value. Probably the presence of extra points on the Savio curve could give more information about bedload transport behavior for the Savio river.

To obtain an estimation of the bedload sediment transport at the measuring sites few of the most well known formula available in literature have been used (i.e. Meyer Peter and Muller, 1948; Meyer Peter and Muller modified by Wong & Parker, 2006; Bagnold, 1980; Martin, 2003; Recking, 2010). A comparison of measured and predicted bedload transport is given for both study cases. It is evident that all formula do not adequately predict bedload deposition along the Fiumi Uniti reach except for Martin (2003) approach which show overall aggradations. Results shows an overestimation of almost all formulae (MPM, 1984; MPM modified by Wong and Parker, 2006; Recking, 2010) at least of one order of magnitude. Concerning MPM (1948) and the modified version (Wong and Parker, 2006), they both over predicts bedload transport on the Fiumi Uniti. In particular the correction introduced by Wong and Parker (2006), reduces the prediction by a factor of about two respect the original version. Results do not univocally suggest that one formula is strongly preferred, except for Martin (2003) and Bagnold (1980) criteria which seems to provide more reasonable predictions than other equations employed. A more detailed analysis underlines that the application Martin (2003) criterion on the Fiumi Uniti dataset seems to show lower discrepancies with the measured data in the range of low flow discharges, while significant ones are visible for higher flow discharges. Related to the Savio river Martin (2003) criterion shows disagreement for the whole dataset. Furthermore Bagnold (1980) equation highly underestimate measured values of bedload transport both on the Fiumi Uniti and on the Savio, even if it seems to suit better for the latter one.

Concerning mid-term bedload discharge reconstruction it is possible to say that, except for the fact that there are some gaps in the available water discharge dataset, results testify the existence of noticeable bedload transport in the mid-term period. Visible gaps are fortunately present in the summer period, i.e. in the dry period where floods more rarely occur. For this reason it is possible to state that the dataset is consistent. The results reveal interesting information related to comparative calculations realized with the three methods performed. The most noticeable element is the reasonable uniformity of the results obtained with methods (a) and (b). In fact, against all expectations, results obtained performing these two methods differs only of a factor around 0.9 for the Fiumi Uniti and 0.7 for the Savio case ($Q_{bb}/\ Q_{ba}$). Therefore, concerning the two studied rivers, despite the fact that the first method used (i.e. method a) is more precise as it relates to real-time data, the second approach can also be considered well applicable for bedload transport estimation purposes as it is only slightly different from the first one. Instead, relating to the estimation realized with Martin (2003) criterion on the long period, it is possible to affirm that results suggest overall a tendency to over predict bedload discharges.

Finely considering the Fiumi Uniti river and the analysis performed on the time of peak T_p (useful to define the ratio between peak and the daily mean discharge) is consistent with the estimated time of concentration T_c, which results around 24 hours. Moreover results show a linear increase of the time of peak T_p with the increase of peak of flow discharge Q_p (Figure 6.15). This increase is expected to saturate, resulting in a asymptotic value, in standard condition of uniform and synchronous rains over the whole basin. In order to make further considerations additional investigations are needed, possibly including a study on precipitation occurred during the most significant flood event. Focusing then on ratio between average daily water discharge $\overline{Q_d}$ and peak of flow discharge Q_p, it is possible to observe a variation of the value between 0.3 and 0.9. Results obtained by $\overline{Q_d}\ /Q_p$ demonstrate a comparable variation already partially noted in the previous analysis ($Q_{bb}/\ Q_{ba}$). Moreover results has permitted to observe that, while ratio of empirical $\overline{Q_d}\ /Q_p$ fluctuates between 0.3 and 0.9, the theoretical parameter reduces the range between 0.4 and 0.8. On this front further analysis are necessary, possibly including examination on additional data.

6.7 Conclusions

The measurement of bedload transport in the Fiumi Uniti and Savio rivers is one of the attempt to quantify bedload transport in the Emilia-Romagna rivers in order to understand the phenomenon of regional coastal erosion. In fact the reduction in river sediment supply and the consequent alteration of the beach sediment budget are among the most important causes of such beach retreat. To approach this problem direct measurement of bedload transport have been carried out with the use of a standard sediment traps (Helley-Smith bedload sampler, US BL-84). A dataset of 24 and 11 flood measurements was obtained. The data show the existence of quite appreciable bedload transport by both rivers with prevailing transport of medium to coarse sand.The analysis also revealed visible variation in bedload transport in both rivers justified by the fact that bedload transport did not occur in the complete active portion of the channel but only in some of its subsections, alternating thus the computation on the total cross-section. On this purpose further investigation are necessary. Moreover, while the Fiumi Uniti bedload dataset seems wide enough to make any consideration, extra future measurement on the Savio river would be appreciable to enlarge its dataset with the possibility to give further information about its behavior in terms of bedload transport.

Among all bedload prediction approached tested, prediction of bedload transport in the studied reach have the tendency to over or under predict bedload transport rates within an order of magnitude for all equation. Only Martin (2003) and Bagnold (1980) approaches provide the best results: bedload prediction are closest to field observations for both rivers cases. Acceptable range of errors is provided by both equations. Not surprisingly, simple stream power correlation proposed by Bagnold (1980) and Martin (2003) offers better results. In fact application of stream power to predict bedload transport rate providing satisfactory agreement has demonstrated to have good results in several applications (Martin 2003; Gomez, 2006; Gao, 2012; Lemma et al., 2019). Concerning mid-term bedload discharge reconstruction it is possible to notice a reasonable uniformity of the results obtained with all tested methods, except for the estimation realized with Martin (2003) criterion which strongly overestimates bedload transport producing results even two times higher than the ones obtained with the other two methods.

Finally considering the analysis performed on the time of concentration on the Fiumi Uniti river, the study demonstrates good estimation of the time of concentration with the SCS-CN method. Results obtained with SCS-CN method are well comparable with results obtained focusing on the detailed study of the most significant flood events. Focusing on these events it has been possible to observe a distinction between empirical and theoretical ratio of average daily water discharge and peak of flow discharge ($\overline{Q_d}/Q_p$), demonstrating that there is a fluctuation of data of 0.3 and 0.9, reduced to 0.4 and 0.8 obtained with theoretical computation. Additional data and investigation are necessary to validate the proposed hypothesis.

7. Summary conclusions

7.1 Main findings of this thesis

The aim of this thesis, as pilot-project for a possible bigger regional scale study, was to enhance data set of river sediment supply to the beaches of the Emilia Romagna region, which in the last decades were affected by a severe erosion. Bedload measurement campaigns carried out in the Fiumi Uniti and Savio river, considered as representative rivers of the region, allowed to obtain significant data related to the sediment size of the bedload, to quantify bedload transport rate for each flood in association with flow discharge, to define incipient motion conditions as well as to understand the influence of moving dunes on flow resistance. Knowledge of all the above mentioned aspects is crucial for the understanding of the complex processes linked with sediment (and bedload) transport and, especially, to quantify its rate for coastal stability, preservation and management.

Measurements carried out a few kilometers upstream of the mouth have shown an appreciable bedload transport capacity in both rivers and a supply to the sea prevalently of medium (and coarse) sand in both cases. The dataset obtained from both rivers evidenced uniformity in bedload transport results. In particular this is visible for the Fiumi Uniti considering both the 2005-06 field campaign and the new data-set of field measurements undertaken in 2017. In order to have a complete overview for the Savio case it is however necessary to integrate the number of field measurements with further campaigns. Well known formulae available in the literature have been used to estimate bedload transport in the studied rivers, obtaining that almost all formulae overestimate or underestimate at least of one order of magnitude. Martin (2003) and Bagnold (1980) approaches are the only ones which seems to return better predictions, with the smallest difference between estimated and measured data. Simple stream power criteria proposed by the above mentioned approaches returned the best results, confirming the satisfactory results obtained by other authors (e.g. Martin 2003; Gomez, 2006; Gao, 2012; Lemma et al., 2019).

Bedload measurement campaigns allowed also to estimate the critical shear stress for bed material incipient motion in these two sand-bed rivers. A comparison with the results of well-known criteria available in literature was carried out in order to predict the threshold conditions for bed particle entrainment. The results obtained pointed out a significant inaccuracy of the literature criteria in predicting critical shear stress values. Results indicate that these criteria largely under predict the value of critical shear stress, whereas others slightly over predict the actual threshold. In this context considering the results related to the Shields dimensionless parameter, all criteria show a big difference in values (i.e. across an order of magnitude), except for Carling criterion which seems to be the most acceptable one. Carling approach assumes irregularly shaped grains and takes into account factors like grain size, grading, packing and protrusion which are completely excluded in the other methods. Despite Carling approach has been developed in steep, narrow (aspect ratio, i.e. width to depth ratio < 11) gravel (or even coarser) bed channels, it seems to be the most suitable approach for the present study cases. Further field investigations are however necessary.

The results obtained on bedload transport and its incipient motion condition have large implications in the assessment of bedload yield in sand-bed rivers. Considering the role of bedforms in affecting flow resistance and grain incipient motion and hence the quantification of bedload yield by means of classical formulas, also this issue was investigated in the field. Considering the approaches already available in literature, a new

model based on 491 field measurements on sand bed rivers with dunes has been realized. Accounting for a different vision of the dune pattern, a revisiting of the Borda theorem complementary to dune geometry and skin roughness models were tested, resulting in a new approach validated with excellent result in sensitivity analysis. To better understand the bedform effect on flow hydraulic in the studied rivers, bathymetric surveys of river bed were carried out through classical echosounding techniques. Bedform geometry was investigated. Furthermore, in order to understand the influence of the presence of moving dunes on flow resistance, fundamental mechanisms of flow resistance (grain and bedform influence) were explored comparing two models available in literature (Engelund and Hansen, 1967, Engelund and Fredsoe 1982) and the one proposed in this study. The tested field conditions show that the bedform drag and the grain roughness contributions are almost equivalent.
Finally, having taken note of all the information obtained until now, an attempt of mid-term reconstruction of bedload transport on both rivers was performed. Results demonstrate positive feedback from all tested methods resulting in an average yearly bedload discharge equal to 1400 ton/year and 1300 ton/year for the Fiumi Uniti and Savio, respectively. The accuracy of the tested methods has been discussed resulting that a reasonable uniformity exists between the use of real data of water discharges and average daily water discharges, while Martin (2003) criterion strongly overestimates bedload transport producing results even two times higher than the ones obtained with the previous ones. The field measurements indicate that the Fiumi Uniti river has higher bedload transport rates than the Savio river. This discrepancy may be due in large part to the difference in the catchment characteristics (the Fiumi Uniti basin is almost two times that of the Savio) and therefore the Fiumi Uniti are subjected to a larger sediment supply and higher stream power. Moreover, the Savio basin has more hydraulic infrastructures (weirs, especially in the headwaters) which may trap part of the bedload in the upstream reaches.
Further deepening realized on the Fiumi Uniti river concerning selected parameters of the flood hydrograph have returned a distinction between empirical and theoretical ratio of average daily water discharge and peak of flow discharge ($\overline{\mathrm{Q_d}}/\mathrm{Q_p}$), demonstrating that the fluctuation of data is reduced with the theoretical computation. Relative to this latter study supplementary investigation are necessary to validate the performed theory.

7.2 Practical implications

7.2.1 *The role of sediment load in the morphodynamic evolution of the rivers and the beaches downstream: hypothesis of sediment processes manipulation at the basin scale*

Morphodynamic evolution of a river mouth and adjacent beaches derives from the interaction of complex and interconnected factors such as fluviatile sediment input, littoral drift, tide and wave power, as well as human activities (e.g. recreational use of the beach for tourism purpose or coastal protections) which interact with each other and affect the evolution of the area. Excluding the marine component for the moment, the littoral system of the Emilia-Romagna region is mostly fed by sand brought to the sea by numerous rivers. Sediment supply of the region is therefore strictly dependent on sediment transport contributions that flow into it. The progressive reduction of fluviatile bedload sediment transport has represented and still represents the main cause of erosion of the beaches of the region. As highlighted in the "Piano Costa 1981", at the end of the 1970s a decrease of 3-4 times bedload transport was registered compared to the conditions of the 1940s (www.ambiente.regione.emilia-romagna.it). Following this evidence, the Emilia-Romagna Region issued legislation aimed at blocking sediment extraction from the riverbeds. The first effect of the legislation in the resumption of sediment transport to sea by some rivers have been perceived clearly on some beaches (i.e. Cattolica, north of the Marecchia estuary and at Goro). For the Fiumi Uniti and the Savio rivers, the analysis did not find a recovery of the sand contributions to neighboring beaches due also of the dense vegetation grown inside the riverbeds (and not removed even for 25-30 years), together with subsidence issues. Previous sediment transport estimates proposed that the sediment transport were next to zero with some positive, even rare, elements highlighted in the Report on the state of the coast in 2000 (Preti M., 2002). The more recent study of this doctoral thesis performed on the Fiumi Uniti and Savio river suggests how the amount of sediment delivered to the Romagna coastal zone might be, despite appreciable. Possible and applicable solutions such as those already implemented (i.e. the block of riverbed mining), could further increase the availability of inland waterway transport. Between these there are the following : the increase in arable land, since more erodible than those currently uncultivated; the shift downstream in the same riverbed of the materials excavated for hydraulic safety reasons and, finally, the removal of the hydraulic works (such as dams) which are no longer used. On this purpose in particular**,** it has been demonstrated by scientific studies that the total volume of sediment trapped behind dams and hydraulic infrastructures reaches globally almost 26% of the total budget (Syvitski et al. 2005b). Considering thus the large quantity of infrastructures present on the studied river basins both in the mountainous and plain areas, the study of their trapping efficiency may be examined. By comparing the sediment flux entering and leaving the reservoirs together with the residency time of sediments and water discharge, it is expected to find some solutions to the problem. As occurred in several other cases, decommissioning dams should probably increase the sediment flux to the coastal zones (Syvitski and Milliman, 2007). Since dams are deteriorating river systems altering sediment and geomorphology of the river channel (Williams and Wolman, 1984; Ligon et. al., 1995) together with negative influence on water quality and jeopardy for river ecosystems, the dam removal trend is worldwide increasing. Thus some benefits in order to increase sediment load to a higher value are not to be excluded with possible dam dismantling within the studied basins.

7.2.2 *Sediment load as an important contribution in support to coastal management*

Beach sediment management in Emilia-Romagna is based on specific Regional Guidelines which essential aims are: 1) beach sediment management for waste reduction of sand resources, i.e. sediment loss from the coastal system; 2) management of coastal accumulations aimed at maintenance of the critical coastal zones with sediments coming from both naturally nourished coastal stretches and accumulations in correspondence with defence works, piers harbour, harbour or river mouths. Emilia-Romagna beach sediment management consists of seasonal maintenance practices essentially based on sand nourishment interventions with material coming from outside the coastal system (mainly from submarine sand deposits) together with realization and maintening of emerged and submerged breakwaters. The knowledge of sediment accumulation of rivers at a seasonal or annual time scale could be an important factor for beach management. The quantification of the amount of sand delivered form rivers, i.e. which can naturally feed the downstream beaches, could be effectively compared to the amount of sand which is necessary for beach preservation. Information on bedload transport capacity from rivers could help coastal managers to plan for a more effective management of beaches: knowledge of the amount of fluviatile sand is in fact essential to estimate the amount of extra sand which has to be delivered from external deposits, maybe giving also the opportunity to coastal managers to save extra money from season to season as needed.

7.3 Main limitations of the study

Limitations of the study are essentially linked with the operation mode and the uncertainties linked to it. Several limitations, already anticipated in the previous chapters, were discussed regarding the use direct techniques, in particular connected to the use of the mechanical samplers as the Helley-Smith. Among them, there is the sampling procedure which requires preliminary notions (flow depth and velocity), and the necessity to sample numerous verticals of a cross-section, making it become a time consuming task. Moreover the heaviness and bulk of the instruments, which necessitate at least three people, makes it costly even from the human resource point of view. Furthermore there are uncertainties linked with the disturbance to flow and bedload transport rate due to the location of the instrument on the streambed. The challenging position of the sampler on the streambed, in combination with the presence of possible bedforms, creating bed irregularities, avoids the appropriate positioning of the instrument. These last two are both examples of possible inaccuracies of taking representative samples. On this purpose, more effort should be spent on trying out new technologies which can improve the quality and accuracy of the collected data, eliminating the above mentioned sources of error. The application of these technologies, as surrogate of conventional techniques, can operate in a continuum and automatic way on the river site providing a solution for consistent, inexpensive, accurate bedload transport measurements with minimum personnel supervision. Among the most renowned are: sonar, radar, smart sensors, hydrophones, impact columns, impact plates and impact pipes. Despite the fact that some of these technologies have been calibrated only in laboratory flume, others already show great potential in both the monitoring laboratory and the field campaigns. A complementary use of conventional techniques (such as the one already used) together with technologically advanced ones could already be a good compromise for future perspective.

7.4 Perspectives for future research

7.4.1 *Future mapping of river channel at high resolution*

Despite still fairly limited in rivers, the combination of terrestrial laser scanning with optical bathymetry of the underwater portion of the reaches, could be a key component of a bigger multidisciplinary approach to understand the geomorphological behavior of the river bed. On this purpose, complementary to what has been reported in this thesis, it would be really interesting to reproduce the downstream reaches of the studied rivers (starting from the measuring sites arriving to the river outlets) with high resolution. For instance the production of a high-quality 3D model of the channel will help to better understand sediment transport processes with a deepening investigation on bedforms and their behavior.

7.4.2 *A combined study of sea storms and river floods to estimate sediment contribution for the coastal zones*

Despite the effort made in this doctoral thesis in quantifying the amount of sediments released by two representative rivers of the Emilia-Romagna region, still a lot of work has to be done on this front. In particular a regional scale method needs to be implemented in order to estimate sediment transport over the entire regional coast. Moreover a combined approach based on the effect of sea storms and river floods would be also necessary aiming at understanding the possible sediment accumulation (or erosion) in the proximity of the river mouth and its adjacent beaches.

Considering the two representative study sites, it is possible to assume that the amount of sediment transported by the rivers is able to reach the mouth, as well as entering into the littoral sediment budget considering the possible interruption that hard structures can imply on sediment exchange between river and beach.The proximity of the bedload measuring sites to the sea (8.5 and 3.5 km for the Fiumi Uniti and Savio, respectively) allows to make this assumption. So that, to what extent the sediment load from the two studied rivers can explain the morphodynamic evolution of the rivers and the beaches downstream it is difficult to say. In a context where the wave regime is of low energy (mean Hs ~ 0.5 m), the tidal range is microtidal and the storm surge is mainly generated by SE winds (Masina and Ciavola, 2011), river bedload sediment transport could be a not negligible component that needs to be considered in terms of beach sediment budget and preservation. A comparison between river floods and sea storm events, supported by pre- and post-event bathymetric surveys would be necessary in order to make some accurate and quantitative conclusions like done in a preliminary way on the Savio by Grottoli et al. (2020). Several could be the available options: 1) sediment supplied by rivers contributed to shape and feed part of the beach (i.e emerge/submerged or protected/unprotected); 2) once reached the outlet, sediment supplied by rivers could stay at the river mouth and partially be transported by the littoral drift or by storm waves after major events.

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