Edoardo Benvenuto PRIZE

Collection of papers

Studia Ligustica 16

Studia Ligustica

Collana di studi on line per l'approfondimento delle tematiche interdisciplinari
riguardanti la storia, le arti e la bibliografia della Liguria

16

Edoardo Benvenuto Prize

Collection of papers

a cura di
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Prefazione

La promozione degli studi e delle ricerche sulla “Scienza e l’Arte del Costruire nel loro sviluppo storico” costituisce l’obiettivo che l’Associazione Edoardo Benvenuto si è posta, fin dalla sua costituzione, l’8 Giugno 1999, allo scopo di onorare la memoria di Edoardo Benvenuto (1940-1998).
Figura di spicco in ambito genovese, ma conosciuto e apprezzato per i suoi scritti e la sua attività di studioso in Italia e all’estero, Benvenuto si era distinto per la sua singolare attitudine a coniugare armonicamente in un contesto unitario il rigore scientifico e la cultura umanistica. La sua attività spaziava infatti dalla scienza delle costruzioni (di cui era professore ordinario) alla filosofia, dalla teologia alla storia della scienza, dalle arti figurative alla musica. Benvenuto non si limitò tuttavia a svolgere un’intensa attività di studio e di ricerca, ma collaborò efficacemente, in prima persona, alla valorizzazione culturale della sua città: in qualità di esperto dell’UNESCO si era adoperato in modo determinante affinché Genova potesse diventare nel 2004 capitale della cultura; nella sua veste di preside di Architettura, incarico che aveva mantenuto per quasi 18 anni, aveva sostenuto con fermezza l’opportunità del trasferimento della facoltà nel centro storico genovese ed era stato il principale artefice della realizzazione di questo obiettivo, con una intuizione che si rivelò ottimale da un punto di vista urbanistico e sociale.
Ma limitando il discorso alle tematiche più specifiche su cui la Associazione, come sopra ricordato, ha voluto focalizzare la sua attenzione, è interessante ricordare che Benvenuto, vinta la cattedra di professore ordinario in Scienza delle costruzioni nel 1974 e chiamato, dal 1975, ad insegnare presso la nuova Facoltà di Architettura di Genova, della quale diviene preside nel 1979, avvia una innovativa impostazione dell’insegnamento e dello studio delle discipline strutturali, introducendo la prospettiva storica come fondamentale chiave di lettura della evoluzione delle conoscenze scientifiche. Tale impostazione era motivata dall’esigenza, che il nuovo impegno presso la facoltà di Architettura gli suggeriva, di trovare un “luogo d’incontro” tra una cultura tecnico-scientifica con cui si era misurato negli anni trascorsi a ingegneria e una cultura diversa, più orientata alle scelte progettuali e al confronto con le scienze umane.
Scrive infatti Benvenuto nell'introduzione alla sua opera maggiore “La scienza delle costruzioni e il suo sviluppo storico” (prima edizione presso Sansoni, Firenze, 1981; riedizione presso Edizioni di Storia e Letteratura, Roma, 2006): «L’ipotesi da me perseguita è appunto che la lettura storica

possa favorire la configurazione di quel “luogo di incontro” di cui prima accennavo. Il contrasto epistemologico tra una scienza “tradizionale”, qual è la meccanica delle strutture, e le altre scienze dallo statuto più duttile cui l’architetto è orientato, ha *in astratto* l’aspetto di una insanabile *contraddizione*, ma se è colto retrospettivamente nel *reale* sviluppo del pensiero scientifico-tecnico e delle soluzioni costruttive, sull’arco della storia dell’architettura, diventa invece il segno di un *rapporto dialettico* che può essere interpretato e vissuto solo “mescolando” i due orizzonti di comprensione o le due “culture”».
L’approccio metodologico inaugurato da Benvenuto con il suo libro ha avuto un seguito significativo in Italia con la promozione di un dottorato di ricerca specifico e la definizione di una linea di ricerca innovativa, che ha visto la partecipazione e il sostegno di numerosi studiosi e docenti. Il riconoscimento a livello internazionale è giunto qualche anno dopo con la pubblicazione dell’altro suo importante trattato, in due volumi, “An introduction to the history of structural mechanics” (Springer Verlag, Berlin - New York, 1991) e degli Atti del primo Symposium “Between Mechanics and Architecture” (Birkhäuser, Basel, 1995), curati da Benvenuto con P. Radelet de Grave. Ho voluto trascrivere le parole di Benvenuto nel brano sopra riportato perché, a mio avviso, chiariscono molto bene quale era il suo progetto culturale di interpretazione del rapporto tra scienza e arte del costruire alla luce della prospettiva storica. L’obiettivo di portare avanti questo progetto e di proseguire sulla strada tracciata è stato alla base della costituzione della Associazione Edoardo Benvenuto, che in questi anni ha conseguito interessanti risultati sviluppando diverse attività quali: organizzazione di convegni nazionali ed internazionali, conferenze, giornate di studio; collaborazione con istituzioni di ricerca nazionali e straniere; promozione della collana editoriale “Between Mechanics and Architecture”; attivazione del portale *Bibliotheca Mechanica-Architectonica*, prima biblioteca digitalizzata “open source” dedicata alla ricerca storica sui testi di meccanica e architettura. Ma forse l’iniziativa più qualificante è stata l’istituzione del *Premio Edoardo Benvenuto*, giunto nel 2019 alla dodicesima edizione, riservato a giovani ricercatori nell’ambito degli studi storici sulla scienza e l’arte del costruire. L’assegnazione del Premio avviene a valle di un esame approfondito dei testi pervenuti alla Associazione ad opera di una Commissione internazionale di esperti. Scopo del presente libro è quello di raccogliere e di presentare gli studi e le pubblicazioni più recenti prodotte dai premiati delle diverse edizioni del Premio Edoardo Benvenuto.

Giovanni Benvenuto

Preface

The promotion of studies and research on the science and art of building in their historical development constitutes the objective that the Edoardo Benvenuto Association has set itself, sins its establishment, in order to honor the memory of Edoardo Benvenuto (1940-1998).
Prominent figure in the genoese context, but known and appreciated for his writings and his activity as a scholar in Italy and abroad, Benvenuto stod out for his unique ability to harmoniously combine in a single context scientific rigor and humanistic culture. His activity in fact ranged from the construction science (of which he was a full professor) to philosophy, from theology to history of science, from figurative arts to music. Benvenuto, however, did not limit himself to carrying out an intense study and research activity, but collaborated effectively, personally, in the cultural enhancement of his city: as a UNESCO expert he had made a decisive effort to ensure that Genoa could become the capital of culture in 2004; in his role as dean of architecture, a position he held for nearly 18 years, he had firmly supported the opportunity of transferring the faculty to the historic center of Genoa and had been the main artificer of the realization of this goal, with an intuition that turned out to be optimal from an urban and social point of view.
But limiting the discussion to the more specific issues on which the Association, as mentioned, wanted to focus its attention, it is interesting to remember that Benvenuto, having won the chair of full professor in construction science in 1974 and called, since 1975, to teach at the new faculty of architecture in Genoa, of which he becomes dean in 1979, initiates an innovative approach to both teaching and studying structural disciplines, introducing the historical perspective as a fundamental key to interpreting the evolution of scientific knowledge. This approach was motivated by the need, which the new commitment at the faculty of architecture suggested to him, to find a "meeting place" between a technical-scientific culture with which he had measured himself in the years spent in engineering and a different culture, more oriented to design choices and to comparison with the human sciences. In fact, Benvenuto writes in the introduction to his major work "The science of construction and its historical development" (first edition at Sansoni, Florence, 1981; re-edition at Edizioni di Storia e Letteratura, Rome, 2006): «The hypothesis pursued by me is precisely that the historical reading can favor the configuration of that "meeting place" I mentioned earlier.

The epistemological contrast between a “traditional” science, which is the mechanics of structures, and the other sciences with a more flexible status to which the architect is oriented, has in the abstract the appearance of a irremediable contradiction, but if it is grasped retrospectively in the real development of scientific-technical thought and constructive solutions, on the arc of the history of architecture, it becomes instead the sign of a dialectical relationship that can be interpreted and lived only by “mixing” the two horizons of understanding or the two “cultures”».
The methodological approach inaugurated by Benvenuto with his book has had a significant following in Italy with a promotion of a research doctorate and the definition of an innovative line of research, which has seen the participation and support of numerous scholars and professors. The international recognition came a few years later with the publication of his other important treatise, in two volumes “An introduction to the history of structural mechanics” (Springer Verlac, Berlin – New York, 1991) and the Proceeding of the first Symposium “Between Mechanics and Architecture” (Birkhauser, Basel, 1995), edited by Benvenuto with P. Radelet de Grave. I wanted to transcribe Benvenuto’s words in the passage above because, in my opinion, they clarify very well what was his cultural project of interpreting the relationship between science and the art of building in the light of the historical perspective. The goal of carrying out this project and continuing on the path traced was the basis of the Edoardo Benvenuto Association, which in recent years has achieved interesting results by developing various activities such as: organization of national and international meetings, conferences, study days; collaborations with national and foreign research institutions; promotion of the editorial series “Between Mechanics and Architecture; activation of the portal *Bibliotheca Mechanica Architectonica*, first “open source” digitized library dedicated to historical research on mechanical and architectural texts. But perhaps the most qualifying initiative was the institution of the Edoardo Benvenuto Prize, arrived in 1919 in its twelfth edition, reserved for young researchers in the field of historical studies on science and the art of building. The awarding of the Prize takes place after an in-depth examination of the texts received by the Association by an international commission of experts. The purpose of this book is to collect and present the most recent studies and publications produced by the winners of the various editions of the Edoardo Benvenuto Prize.

Giovanni Benvenuto

Antonio Becchi

Il doppio mondo di un "uomo sensibile e immaginoso"

> All'uomo sensibile e immaginoso, che viva, come io sono vissuto gran tempo, sentendo di continuo ed immaginando, il mondo e gli oggetti sono in certo modo doppi. Egli vedrà cogli occhi una torre, una campagna; udrà cogli orecchi un suono d'una campana; e nel tempo stesso coll'immaginazione vedrà un'altra torre, un'altra campagna, udrà un altro suono. In questo secondo genere di obbietti sta tutto il bello e il piacevole delle cose. Trista quella vita (ed è pur tale la vita comunemente) che non vede, non ode, non sente se non che oggetti semplici, quelli soli di cui gli occhi, gli orecchi e gli altri sentimenti ricevono la sensazione.
>
> G. Leopardi, *Zibaldone*, 30 novembre 1828, prima domenica d'Avvento

Un album fotografico, di quelli che custodiscono le immagini più care di un nucleo famigliare. Così potrebbe essere definito questo volume che raccoglie gli scritti della maggior parte di coloro che hanno vinto o sono stati segnalati con una menzione speciale nell'ambito del Premio Edoardo Benvenuto (edizioni 2002-2019). Un album nel quale emerge la varietà degli approcci, dei temi, degli interessi di ricerca, così come in una grande famiglia si distinguono i volti, gli ambienti, gli sfondi che caratterizzano le diverse personalità e generazioni, in atmosfere pervase da un'"aria di casa", che l'album fotografico esalta e custodisce.
Ci si potrebbe chiedere quale filo rosso leghi un saggio sulla *Experimental and numerical response of dry-joint masonry arches subject to large support displacements* a quello sulla *'Accademia della Vachia' in Florence, 1661-1662* oppure *La Coupe des voûtes à la française* al *Reinforced Concrete in Italy* e quali criteri abbiano guidato le diverse commissioni del Premio Edoardo Benvenuto, grazie alle quali si è formato un gruppo di studiosi che oggi riempie d'orgoglio coloro che il Premio hanno fondato più di 20 anni or sono. La vita e l'opera di Edoardo Benvenuto (1940-1998) sono sempre state al centro dell'attenzione di chi doveva valutare il lavoro dei singoli concorrenti, ad esse bisogna dunque tornare per capire l'importanza e il significato di quello che è qui raccolto. Una serie di fotogrammi tratti dalla sua operosissima esistenza può offrirci un'utile chiave di lettura dei

testi presentati in questo volume e del loro promettente "essere insieme", pur nelle evidenti differenze di stile, di forma, di contenuto.
Nato a Genova l'11 dicembre 1940 Edoardo mostrò sin da ragazzo doti fuori dal comune: arte, storia, filosofia, musica, scienze fisico-matematiche e quant'altro offriva la scuola trovarono in lui un terreno fertile, favorito da un clima famigliare profondamente imbevuto di cultura e di impegno politico e sociale. Come forse è accaduto anche a molti dei vincitori del Premio a lui dedicato, a Edoardo la scelta universitaria, che seguiva un brillante esame di maturità, riuscì difficile. Animato da mille interessi e curiosità non effimere, dotato di un solido bagaglio di conoscenze di base che andavano ben al di là della preparazione di un liceale studioso e appassionato, Edoardo non sapeva quale strada scegliere e alla fine optò per la Facoltà di Ingegneria, sotto l'influsso di Giuliano Durante (1915-1992), ingegnere capo del Comune di Genova, zio materno amato e ammirato. Come lui stesso ricorderà più di trent'anni dopo in un breve profilo autobiografico (lettera indirizzata al prof. Francesco Palma, direttore della rivista *Professione ingegnere*, 18 luglio 1995): «Ciò nonostante, scelsi infine, seppur con qualche tormento interiore, la Facoltà di Ingegneria. [...] Come avrei potuto pretendere di dire qualcosa di sensato in filosofia, ignorando l'universo della tecnica che caratterizza l'epoca attuale?» Si trattava, quindi, di far maturare quella che lo stesso Edoardo definiva «una viva passione, ed anzi un'irrefrenabile furia, per la speculazione filosofica (mio padre era un insigne pedagogista), non disgiunta da un costante interessamento per le arti»" (*ibidem*).
La scelta di ingegneria non gli impedisce di diplomarsi in pianoforte nel 1962, sotto la guida del Maestro Louis Cortese (1899-1976, musicista di grande talento e direttore del Liceo musicale Niccolò Paganini di Genova, laureatosi in matematica nel 1924), sempre coniugando le arti con le scienze. Edoardo non vedeva in quel connubio una giustapposizione di diversi ambiti disciplinari, semmai l'esito naturale e necessario di riflessioni radicali, del quotidiano interrogarsi sul come e sul perché dell'esistenza, trattando con eguale intensità d'animo la *firmitas*, l'*utilitas* e la *venustas* del nostro breve soggiorno terreno. In questo percorso di studio e di ricerca la filosofia e la teologia assumevano un ruolo guida, perché da esse era per lui inevitabile partire e ad esse ritornare ogni qual volta i pensieri volgessero al nocciolo duro della loro origine, al magma esistenziale nel quale essi prendevano forma.
Questo modo di affrontare l'impegno intellettuale lo fece immediatamente distinguere negli studi universitari, durante i quali ebbe la fortuna

d'incontrare giovani colleghi di straordinario valore, come Piero Villaggio (1932-2014) e Alfredo Corsanego (1936-2008), e un Maestro d'eccezione, Riccardo Baldacci (1917-1986), che di Edoardo colse al volo il talento e le potenzialità. Fu Baldacci a mandarlo in missione alla Facoltà di architettura, riconoscendo in lui l'uomo del dialogo, della ricerca di frontiera, della scienza e della tecnica messe al servizio del bello e dello slancio creativo. «Maledetti architetti, benedetti architetti», amava dire un altro ingegnere *sui generis*, Giulio Pizzetti (1915-1990), che volentieri si mescolava tra le file di quel popolo di eterogenea natura. Sin dai primi anni trascorsi ad Architettura Edoardo condivise quello che recentemente Jacques Heyman ha ricordato nella sua autobiografia, facendo riferimento all'esperienza di insegnamento ad Harvard negli anni Sessanta: «students of architecture are among the brightest I have taught» (Heyman, 2022, p. 75).
Nella tesi di laurea Edoardo ebbe modo di occuparsi di urbanistica, con un progetto dedicato all'inserimento di una Facoltà universitaria nel centro storico di Genova, presagendo, forse, quello che poi gli capitò di realizzare, da protagonista, 25 anni più tardi, guidando una delle più riuscite ed imponenti operazioni urbanistiche del Dopoguerra genovese, ossia il trasferimento della Facoltà di architettura dal colle di Albaro a quello di Sarzano, in pieno centro storico. Operazione felicissima, della quale andava giustamente orgoglioso e nella quale mise in evidenza, una volta di più, le sue doti "politiche", la sua passione per la città e il bene comune.
La riflessione storica, in tutte le sue forme e declinazioni, era la palestra dei suoi interessi di ricerca. In quell'humus a lui congeniale sin dall'adolescenza, la scienza delle costruzioni prese un posto singolare. Usualmente temuta se non odiata dalla maggioranza degli allievi architetti, poco abituati alla ginnastica fisico-matematica applicata alle costruzioni, nelle mani di Edoardo quella materia si trasformava in uno strumento formidabile per capire l'arte del costruire, le scienze applicate alla costruzione, i molteplici saperi che nel cantiere trovano spesso sintesi inaspettate. Dalle lezioni ad Architettura nasce il volume *La scienza delle costruzioni e il suo sviluppo storico*[1] (1981), che nel 1991 sarà radicalmente rivisto e riscritto nell'opera *An Introduction to the History of Structural Mechanics* (Benvenuto,

[1] Benvenuto, 1981. Lo stesso volume è stato ristampato nel 2006 e nel 2010 presso le *Edizioni di Storia e Letteratura* (Roma), nella serie *Between Mechanics and Architecture* (<www.storiaeletteratura.it>). È attualmente in corso una terza ristampa.

1991). Nel frattempo continuava il suo impegno come docente, preside di Facoltà, per un certo periodo pro-rettore all'edilizia universitaria, ma anche come teologo militante. Il mondo degli architetti gli dava la possibilità di confrontarsi ogni giorno con gli aspetti più disparati del sapere, dalla storia delle scienze alle tecniche costruttive d'avanguardia, dal progetto architettonico alla saggezza racchiusa nelle antiche regole dell'arte.
Nei primi anni Ottanta Edoardo si rende conto che le Facoltà di architettura hanno bisogno di un radicale ripensamento del loro modo di strutturare la didattica e la ricerca. Per questo si impegna nelle commissioni di riforma a livello ministeriale e, appena se ne presenta l'occasione, fonda con due colleghi, Salvatore Di Pasquale (1931-2004) e Antonino Giuffrè (1933-1997), un dottorato di ricerca che ha l'intento di formare una nuova generazione di docenti e ricercatori capaci di studiare e insegnare la storia delle scienze e delle tecniche costruttive. In quel nuovo vivaio accademico le usuali suddivisioni disciplinari vengono a cadere, in nome di una sintesi culturale di più ampio respiro, che coinvolge la scienza e la tecnica delle costruzioni, la storia dell'architettura, delle scienze e delle tecniche, la filologia e la filosofia, insieme a molto altro ancora. Per lui quel dottorato, basato sull'alleanza di tre sedi universitarie (Genova, Firenze e Roma), deve modificare per sempre il ruolo della scienza e della tecnica delle costruzioni nelle Facoltà di architettura e, se possibile, di ingegneria: non più soltanto palestre di calcolo più o meno raffinato, ma finalmente spazi del sapere dove l'anima politecnica può svilupparsi al meglio, tra arte e scienza del costruire, sempre indagandone lo sviluppo storico per comprendere le costruzioni del passato e innovare quelle del presente.
La svolta auspicata da Edoardo e dai suoi colleghi di cordata non ebbe luogo, se non in minima parte. I motivi furono molteplici e il tenore del dibattito può essere descritto con le parole che Alfredo Corsanego usò nel raffigurare certe tenzoni scientifiche rievocate nelle lezioni di Riccardo Baldacci: «ora scontri tra guerrieri omerici, ora più modesti certami da *chanson de geste*, talvolta, ahimè, inutile armeggiare di chi, come quel personaggio ariostesco, 'credeva di pugnare ed era morto'» (Corsanego, 1989, p. XVIII). Arroccamenti disciplinari, peraltro ampiamente prevedibili, gelosie concorsuali, inettitudine e insipienza di alcuni docenti e discenti di non chiara fama, scarsità di talenti che fossero all'altezza di quella svolta (coraggiosa quanto ambiziosa), finirono per favorire un silente ma inesorabile ritorno all'ordine, venato da poche ma comunque significative voci fuori dal coro. Quel tipo di approccio storico-critico alla scienza e all'arte del costruire ebbe maggior successo all'estero, dove trovò molti seguaci

nell'ambito del movimento *Construction History* (*CH*), soprattutto dopo che Edoardo, insieme a Patricia Radelet de Grave, lanciò il progetto *Between Mechanics and Architecture* (Radelet de Grave & Benvenuto, 1995) nel simposio omonimo organizzato all'interno del XIX congresso internazionale di storia della scienza (Saragozza, 1993).
All'arte e alla storia del costruire la scuola genovese, formatasi intorno all'insegnamento di Edoardo, ha dedicato numerosi seminari a partire dal 1999 e nel periodo 2000-2004 ha cercato di coordinare le iniziative italiane intorno a questi temi. Il 6 dicembre 2002 si è svolto a Genova il seminario *La storia del costruire in Italia: didattica e ricerca*, che il 28-29 novembre 2003 ha avuto un seguito con l'incontro internazionale *La storia del costruire in Europa: collaborazioni e prospettive di ricerca*. Nello stesso anno l'Associazione Edoardo Benvenuto (AEB) ha partecipato come co-promotrice al Primo convegno internazionale di storia del costruire, organizzato a Madrid dagli amici e colleghi spagnoli. L'anno successivo viene pubblicato dalla AEB il primo resoconto internazionale sulla *CH*[2] e nel 2007 viene lanciato il progetto *Bibliotheca Mechanico-Architectonica*[3]. Queste iniziative si intrecciano con la nascita del Premio Edoardo Benvenuto nel 2002 e ne costituiscono il necessario quadro di riferimento.
A distanza di 25 anni dalla morte di Edoardo il bilancio può lasciare perplessi o entusiasmare, a seconda dei punti di vista. A livello internazionale si moltiplicano le iniziative che mostrano una chiara sintonia con il progetto culturale che egli aveva coltivato, ma sul versante accademico italiano la ricerca e la didattica pertinenti alla *Scienza delle costruzioni e il suo sviluppo storico* hanno assunto un carattere marginale e sono affidate alle cure di pochi volenterosi che ancora credono nell'efficacia e necessità dell'approccio storico-critico rivolto alle scienze per l'architettura. Nel più vasto campo della *CH* il contributo italiano risulta di prim'ordine ma sfilacciato, come dimostra il fatto che l'Italia non ha ancora un'associazione nazionale ad essa dedicata, al contrario di Germania, Francia, Portogallo, Regno Unito, Spagna e Stati Uniti. L'Italia, inoltre, non ha mai ospitato un congresso internazionale promosso da quel network di studiosi interessati alla *CH* che ogni tre anni si dà appuntamento in una sede di prestigio, come è avvenuto in Spagna (2003), Regno Unito (2006), Germania (2009), Francia

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[2] Becchi *et al.*, 2004. Sullo stesso tema anche Becchi *et al.*, 2018.

[3] Cfr. <www.bma-project.org>.

(2012), Stati Uniti (2015), Belgio (2018) e Portogallo (2021). A questi Paesi deve associarsi l'Algeria, che nel giugno 2021 ha organizzato a Tlemcen il primo convegno sulla storia del costruire in terra africana. L'assenza dell'Italia risulta sorprendente se si pensa al numero e alla qualità degli studiosi italiani che operano in questo campo di ricerca e al fatto che molti di essi sono stati tra i primi a sostenerne lo sviluppo.
In quest'inizio di primavera del 2023, trent'anni dopo il simposio *Between Mechanics and Architecture* di Saragozza e nel ventesimo anniversario del primo congresso internazionale di Madrid, le attività del *Construction History Group* (Politecnico di Torino), fondato nel giugno 2020, hanno posto le basi per l'organizzazione del IX congresso internazionale di storia del costruire in Italia nel 2027, dopo quello che si terrà a Zurigo nel 2024. Quell'evento segnerà un significativo ritorno alle origini per il filone di ricerche che Edoardo aveva contribuito a promuovere: negli stessi anni nei quali egli sviluppava a Genova e in Europa le ricerche sull'arte del costruire, a Torino lavoravano al Politecnico amici e colleghi di eccezionale valore - Augusto Cavallari Murat (1911-1989), Giulio Pizzetti (1915-1990), Roberto Gabetti (1925-2000), Anna Maria Zorgno Trisciuoglio (1939-1999), Luciano Re (1939-2019), Franco Rosso (1939-2019), Vittorio Nascè, Carlo Olmo e tanti altri - che con Edoardo condividevano interessi di ricerca e visione umanistica ad ampio raggio. L'interesse storico di tradizione politecnica affondava le sue radici in anni lontani, ad esempio negli studi pionieristici di Carlo Promis (1808-1873) e in quelli di Giuseppe Albenga (1882-1957), che insegnò *Costruzione di ponti* e fu anche rettore del Politecnico di Torino (1929-1932). Albenga aveva coltivato precisi interessi storici relativi alla scienza e tecnica del costruire, seguiti con grande attenzione da Riccardo Baldacci, che a lui dedicò il trattato *Scienza delle costruzioni* (Baldacci, 1970): «Alla cara Memoria di GIUSEPPE ALBENGA Ingegnere e Maestro». Ai suoi studi Baldacci fa riferimento nella *Presentazione* del volume *La scienza delle costruzioni e il suo sviluppo storico* (1981), stabilendo un significativo collegamento "aristotelico" tra la scuola torinese e quella genovese, tra il vecchio Maestro e il giovane allievo, nel ricordo di «un indimenticabile e insigne nostro maestro, Giuseppe Albenga, che, proprio da una sua originale interpretazione filologica di un passo dei *Μηχανικά Προβλήματα*, attribuiva a quest'opera il merito prioritario di aver intuito il concetto di flessione»[4].

[4] Baldacci, 1981, p. VI, con nota a piè di pagina che rinvia allo scritto *Il problema della*

I saggi qui presentati, frutto delle prime 12 edizioni del Premio Edoardo Benvenuto, devono essere letti sullo sfondo di queste vicende, avendo ben presenti i tratti salienti della sfida culturale che Edoardo aveva lanciato. Le carriere e la salutare diaspora intellettuale dei giovani premiati dimostrano che i commissari chiamati a selezionare i migliori candidati avevano visto giusto: Cambridge (GB e USA), Parigi, Louvain la Neuve, Berlino, Bonn, Potsdam, Parma, Venezia, Roma, Napoli, Madrid, Douliou City... sono le città associate ai giovani vincitori, che in seguito hanno affinato le ricerche e il percorso di formazione nei campi a loro più congeniali. All'inizio il Premio Edoardo Benvenuto è forse servito ad alcuni di loro per progredire rapidamente nella carriera accademica, ora è il Premio stesso a godere di luce riflessa, in angoli del mondo dove probabilmente la *buona novella* di Edoardo non sarebbe mai arrivata. Questo successo lo dobbiamo anche a coloro che, nell'arco di un ventennio, hanno fatto parte delle commissioni giudicatrici. Essi hanno saputo selezionare i vincitori con occhio linceo e per questo desideriamo qui ricordarli con profonda gratitudine: Bill Addis, Stefano Bennati, Silvia Briccoli Bati, Massimo Corradi, Alfredo Corsanego (1936-2008), Salvatore D'Agostino, Luigi Gambarotta, Rolf Gerhardt, Riccardo Gulli, Jacques Heyman, Santiago Huerta, Karl-Eugen Kurrer, Sergio Poretti (1944-2017), Patricia Radelet de Grave, Joël Sakarovitch (1949-2014), Anna Sinopoli. A questo aureo gruppo di colleghe e colleghi va il merito di aver saputo individuare i migliori talenti, dando al Premio l'autorevolezza e la *bona fama* che la comunità scientifica gli riconosce.
Speriamo che l'album di famiglia qui presentato possa continuare ad essere sfogliato ancora a lungo e, se possibile, arricchito di nuovi contributi di generazione in generazione, volgendo lo sguardo con riconoscenza alla vita e all'opera di Edoardo Benvenuto, "uomo sensibile e immaginoso."

flessione della trave elastica ed il suo evolversi (Albenga 1956). Nella nota questo contributo dello studioso piemontese è datato 1957, ma la prima pubblicazione risale all'anno precedente. Albenga presentò la sua memoria all'Accademia delle Scienze di Torino il 20 giugno 1956 e morì pochi mesi dopo, il 19 gennaio 1957. I *Μηχανικά Προβλήματα* (*Problemi meccanici*) di scuola aristotelica, per lungo tempo attribuiti allo stesso Aristotele, saranno non a caso oggetto di due ricerche pubblicate decenni più tardi nell'alveo del progetto *Between Mechanics and Architecture*. Cfr. Becchi, 2004 e 2017.

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Edoardo Benvenuto Prize
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Pierre Smars

Safety analysis of masonry arch and vaults with a note on the kinematic safety of arches*

In the first section of the paper, the main results of a PhD thesis defended by the author in 2000 about the stability of masonry arches and vaults are presented. The theoretical approach is completed and collated with field observations made in a specific context: the Gothic vaults of the former Duchy of Brabant in Belgium. The central question treated is *model uncertainty*. Modelling hypotheses are often taken because they are assumed safe *a-priori*. It is nevertheless shown in the thesis that it is not always the case. In particular, a higher tension resistance of the material sometimes leads to weaker structures. This disturbing fact is one more reason stressing the importance to combine mathematical modelling with other approaches: analysis of the construction techniques, of the pathology, of the history of construction and deformations. Doing so may reduce uncertainty and improve confidence in the diagnosis, leading to better decisions. The thesis explores theoretically the influence of *tension*, *friction* and *large deformations* on safety assessment and gathers a large number of observations made in the field about arches and vaults.
In a second section of the paper, one of the topics of the thesis is presented in greater detail: the influence of large displacements of the abutments of arch structures on their stability. The original results of the thesis are completed with new observations. A series of typical deformation scenarios are studied (horizontal spreading, settlement of abutments, complex deformation paths). A phenomenological description is first given on the basis of experiments on a scale model. A range of behaviours is documented; in some circumstances, a significant spread in the results is observed. The same scenarios are then studied using a mathematical model and computer software tools. The process of hinge formation, hinge pattern transitions and the origin of the variations in the results are discussed and clarified.

* The paper was written in 2014. Since then, some topics discussed in the text have seen interesting developments, in particular on the effects of finite displacement of the abutments of arches. Text and references were left in their original form.

1. *Studies on the stability of arches and vaults*

1.1. *Context*

April 26th 2000, the author successfully defended a PhD thesis at the *Katholieke Universiteit Leuven* (Belgium). The dissertation, titled "Etudes sur la stabilité des arcs et voûtes - Confrontation des méthodes de l'analyse limite aux voûtes gothiques en Brabant" (Smars 2000) was written under the supervision of Professor Salvatore Di Pasquale (*Università degli Studi di Firenze* then *Università degli Studi di Catania* - Italy) and Professor Guido De Roeck (*Katholieke Universiteit Leuven* - Belgium). In 2002, the dissertation was awarded the *Premio Edoardo Benvenuto*.
Like many before him, the author was – and still is – fascinated by *Gothic vaults*. It is an exciting object of study: their design, construction and analysis is an *ars* seasoned with *scientia*, a fact acknowledged by most practitioners… who may nevertheless disagree on the meaning of the terms. It is also a rather *hackneyed subject*, where daydreams and references to *auctoritas* are all too common.
In an attempt to avoid these pitfalls, it appeared important to relate any theoretical attempt with direct observations, made in a precise and well-defined context. Geographical and temporal limits were given: two of the four "quarters" of the ancient Duchy of Brabant (Belgium) and the Gothic period (13th-16th century in the area investigated). All vaults considered were built either completely in stone masonry, or in bricks with stone ribs. Most of them were found in the around one hundred Gothic churches of the area.
From the point of view of structural theory, the long interest in Gothic vaults resulted in substantial contributions leading to a better understanding of their behaviour. These contributions are unveiling a distinct – and foreseeable – pattern. As new methods or paradigms of structural design and analysis appear, new types of *models* are used to study vaults, clarifying our understanding and justifying or discrediting previous ideas. A subjective and non-exhaustive list of theories, techniques and some of their famous proponents may include:

- Geometric rules – Jean Mignot, Martínez de Aranda, Rodrigo Gil de Hontañon…
- Catenary (physical models) - Hooke, Poleni, Gaudí…
- Graphostatic - Moseley, Méry…

- Deformation methods (elasticity, finite element analysis…) – Castigliano, Mark, Barthel…
- Limit analysis - Kooharian, Heyman…

Each of these approaches was used to design new structures. Our built environment bears witness to their effectiveness (Huerta in 1990 gave an illuminating view on the often decried geometric rules).
The critical point with structural theories is the common difficulty to test hypotheses (i.e. to verify or falsify them) related not to *design* but to *analysis or assessment of an existing structure* (behaviour or safety). Science is trying to infer general relations from the repetition of specific events. Experimentation is one of its tools, their repeatability a token of legitimacy.
A large corpus of observations about the behaviour of Gothic vaults exists. New observations are also waiting to be made by curious investigators. The numerous existing buildings, ruins, historical accounts of failures, results of deformation monitoring, various kinds of testing… all form a very rich investigation material.
Some observations are sufficiently corroborated to lead to general rules accepted by most. It is for instance recognised that a vault presenting cracks is not necessarily in danger. Numerous standing buildings demonstrate it and theories let us apprehend why.
The question is less clear when predictions on an existing structure need to be made. For an engineer (and possibly for a society interested in the protection of its cultural heritage), the most important prediction to make is to estimate the likelihood of collapse (or of minor failure), i.e. to estimate structural safety. ISCARSAH, the structure committee of ICOMOS is making it a prerequisite to any intervention: «Before making a decision on structural intervention it is indispensable to determine first the causes of damage and decay, and then to evaluate the safety level of the structure». (ICOMOS, 2003).
Unfortunately, the adequacy of theory to model the “real” world is here difficult to investigate. What is a good theory? A successful intervention? How do we measure it? Does it validate a theory? Can the theory be validated? Can it be invalidated? Does it happen often? The lack of repeatability hinders attempts of induction. The lack or crudeness of observables hinders attempts of demonstration.
The fact is that each structure is specific, and variations between them are significant, often not identified, rarely quantified and sometimes not even

practically measurable. Sure, uncertainty is present in any kind of modelling, but it is certainly a key factor when dealing with historic structures.
This is also for this reason that it was decided to limit the investigations to a specific context: in order to reduce and better circumscribe these variations. The *ancient duchy of Brabant* is an area rich in Gothic churches, close to the place of residence of the author and which could be explored systematically and repeatedly. Direct participation in detailed studies on some churches of the area (mainly Saint-James and Our Lady in Leuven) also fuelled our interest and experience.
From Jean Mignot onward, analysts perceived very well "faults" made by their predecessors or colleagues but then too often show a great confidence in the power of their approach. In 1934, Pol Abraham (1934, p. 19) was claiming that «avec un peu d'attention et de sens de la géometrie dans l'espace l'expérience idéale de la bille permettra toujours d'analyser le «fonctionnement» des voûtes usuelles, même complexes». More recent examples of great confidence are not difficult to find. It is our opinion that *uncertainty* and *lack of verifiability or refutability* should prompt to more humility and deserve a discussion in their own rights.
The proponents of *limit analysis* are well aware of uncertainty. Clear and convincing accounts of the question can for instance be found in the work of Heyman (1966) and Mainstone (1997). The author was educated in this tradition.
Nevertheless, some confusion about the foundation of limit analysis arose in our mind. Doubts about their soundness occurred at the occasion of a consultancy on a church in which a few voussoirs of one of the vaults had recently fallen down. An analysis showed that it was possible to find a state of stress in equilibrium and everywhere admissible. Classical plasticity theory, as it is usually applied to masonry structures, predicted therefore that the vault was safe. Nevertheless, that was clearly not the case. In this specific instance, the problem was of a local nature but this kind of explanation was not deemed sufficient. What was wrong with the model? What limits should be given to confidence?
Soon, it appeared that the root of the problem was *sliding*: the fallen voussoirs had slid out of their positions, something which is not supposed to happen when classical plasticity theory is used to analyse masonry structures. Other examples were soon found in the geographical area under investigation. Occurrences are not common but by no means exceptional. The willingness to clear up the confusion became the main incentive behind the thesis. An attitude of *methodological scepticism* was taken to-

wards the two most common paradigms used today: *limit analysis* and *deformation methods* (FEA and others). Doubts cannot be cast on every single aspect of a theory. A limited number of roads were followed, studying the influence of:

- a small (but not null) resistance to tension in the joints
- a limited resistance to friction in the joints
- large deformations of the abutments of an arch
- specific construction techniques (using the vaults in Brabant as a reference)

The discussion occurred at three levels —in three parts— from a general and theoretical discussion about modelling and plasticity to a specific and more practical discussion of existing masonry vaults.

1.2. *Models*

It is well recognised that mathematical models alone have their limits (ICOMOS, 2003; ISO, 2010). Decisions are often taken integrating information of very different origins and natures (another reason behind our decision to describe types and construction techniques in the third part of the thesis). This integration is often difficult, and the rationality of the approach is not always clear and, formally, it is seldom done consistently.
In the first part of the thesis, an attempt is made to give a common framework to deal with this diversity of sources of information. The tools actually used to bring understanding and make predictions are always *models*: mathematical models, experimental models (results from the monitoring of the deformations for instance), judgements of experts, visual observations, historical models… Humans use models to apprehend reality. They are a manageable and partial view of an infinitely complex problem. They are partial (simplifications) because they only consider a finite number of dimensions and parameters and a finite number of logical relations between them.
A formal definition of models is given, which can accommodate very diverse approaches and give a common ground for their integration. Meaning is assigned to models and through them to a “real situation” by trying to compare them with some references: which are also models…
That is another of their limitations, they cannot be valued in an absolute sense.

There is a place for rationality and deductions in the process but there is always a core of basic assumptions rebel to proofs. Taking decisions will always require a level of trust or faith. The recourse to a large variety of models is somehow paralleled with the research of pieces of circumstantial evidence in criminal trials: redundancy gives confidence. In “hard” science redundancy comes through experiments, in the structural analysis of ancient buildings and other fields it comes through the multiplication of models.

In the field of engineering, the limits of an approach are found at the occasions of failures. Those are opportunities to gain a greater confidence after losing it.

A specific discussion about *limit analysis* follows this general discussion about models. In the framework of masonry structure analysis, it is “classically” assumed that:

- The resistance of masonry to tension is null
- There is no sliding between the masonry elements

The critical point is that it is assumed that these postulates are “safe”. If a model of the structure made of such a material is safe, it is assumed that the “real” structure is also safe. The implicit and “wrong” line of reasoning is probably the following: (1) materials having these characteristics have a so-called “standard” behaviour and can be studied with classical plasticity theory, (2) consequently, it can be proved that to reinforce locally a structure will make it safer, (3) if a structure is safer when it is locally stronger that means that it is safe to assume that masonry does not resist to tension. This is of course circular reasoning.

Classical plasticity theory classifies situations as either *stable*, either *unstable*. Non-standard plasticity complicates the matter by adding a new type of situation in which it is impossible to decide whether the structure is stable or unstable. The three categories are:

- Certainly safe
- Possibly safe
- Certainly unsafe

A structure is qualified as *safe* if we trust that the expected changes (extra loads, deformations) will be resisted. Three domains corresponding to the categories above are defined. The extension of the domain of possible or “contingent” stability depends not only on the structure itself but also

on our level of knowledge (or beliefs) about it. To some extent, it can be reduced by further studies.
The possibility of sliding, at the origin of our investigations, illustrates the process. At the level of a joint in the masonry, sliding is prevented if the normal force on the joint is sufficient. If a lower bound can be found, a lower bound can also be computed for the admissible tangential force. This knowledge sometimes may be sufficient to ascertain that a structure is stable. If the lower bound is null (a common situation in statically indeterminate structures), it may just be stated that the structure is potentially stable.
Obviously, any standing structure must be stable to some extent. The subtlety is that, given our level of knowledge, if it is qualified as potentially stable, it is not possible to rule out that, even a very small perturbation, is not going to lead some part of the structure to collapse. Before studying a situation, this domain is very vast. After extensive studies, it will be significantly reduced. It will never be annihilated.
Let us return to the question of finding a lower-bound for the normal forces acting in the joints of the masonry (Smars, 2008). To be sure that some part P of a structure S is safe and will not collapse, it will be necessary to study the structure S-P. Briefly stated, if S-P does not need P and if kinematic conditions allow P to slide, this occurrence cannot be ruled out. That is what happened in the vault discussed above. The joints of the fallen stones were thick and in bad condition. They could not retain the stones. Being thin and not cut as real voussoirs, the stones slid and fell. The structure S-P remained stable.
If S-P needs P to stand, their stability is interdependent. To find the smallest normal forces acting on P, the bounds on the forces necessary to keep S-P stable have to be found. This process requires the use of upper limits of the material resistance. If S-P is stronger, it demands less from P. In that respect, a higher resistance – to tension or compression – in the masonry can be detrimental.
The size of the parts P considered in the analysis has a direct influence on the extent of the domain of *contingent stability*. If P is large, the domain can be small. If P is small the problem becomes intractable. One can never be sure that some grain of sand is not going to “collapse”!
The real structure cannot be studied (a Laplace daemon’s task). But the multiplication of models, observation of the pathology, knowledge about the construction technique, accurate surveys can help us to build trustworthy hypotheses and therefore reduce the domain.

1.3 *Arches*

Arches are the quintessential masonry elements. Their shape and construction technique are dictated by the characteristics of the material and by their discrete nature. Their study is especially instructive. Typical behaviour of complex structures can often be identified, and more easily understood, by studying simpler structural elements.
This part of the thesis begins with the development of a mathematical model of the structural behaviour of arches. The safety domains defined in the previous part are built, first at the level of the joints between the blocks and then at the level of the whole structure, following a line opened a long time ago by Durand-Claye (1867; 1880). Since their 2008 versions, the software tools may be used to study the influence of tension resistance and sliding.
The model is then used to study a specific problem: the influence of large displacements of the abutments on the safety of arches.
Indeed, a common criticism made about to limit analysis is that the method does not address problems related to deformations: structures are considered rigid. This is the strength of methods like FEA; theoretically able to study structures subjected to large deformations and predict the evolution of their safety. The fact is nevertheless that, very often, large displacements are the result of settlements of foundations whose intensity is difficult to predict. This problem is directly related to the question of uncertainty discussed above, an uncertainty which can be reduced by the use of other models. More specifically, the monitoring of the displacement can give a direct and more reliable image of the deformation of a structure. What monitoring cannot do is decide whether an observed deformation is dangerous or not. To assess the safety of the structure in a given configuration, it is necessary to use mathematical models.
The question is further discussed in **section 2**.

1.4. *Vaults*

The two first parts of the thesis were not making any reference to existing structures. The aim of this part, treating of vaults was to gather observations which could be paralleled to the theoretical observations. It gives a better feeling about the importance to give to some behaviours.
Many features are shared by all the vaults produced during the Gothic period in Europe but there is also a tremendous diversity. Numerous factors are at play: materials availability, cultural habits, skills of the builders,

requirements of the sponsors… It appeared to the author that the most reasonable approach was to limit the study to a smaller and more manageable context, i.e.:

- Masonry vaults built in stones or bricks;
- In churches (with a few exceptions);
- In the former Duchy of Brabant (in nowadays Belgium);
- Built during the Gothic period: 13^{th} to 16^{th} century (in the area under consideration)

Systematic visits were organised in the study area and typologies of architectural elements (ribs, webs, flying buttresses…) and pathological observations were prepared.
One of the outcomes of the resulting inventory is a better understanding of the evolution of the construction techniques used in the area. The first motor of this evolution is the attempt to reduce the importance of centring and the growing rarity of the stones which were used for vault construction. The results of this research were also presented elsewhere (Smars, 2009).
Corroborating the two first parts of the thesis, examples of the sliding of masonry elements (often ribs' voussoirs) were discovered in a few churches. Often metallic elements were used as a measure to prevent local collapses.

2. *Large deformations*

One of the topics of the thesis is now presented in greater detail: the stability of an arch subjected to large displacements of its abutments.
This topic was chosen because recent research gives new light on the matter and somehow alter the practical conclusions drawn from the year 2000 analysis.
The idea that the collapse of arches may result from large displacement of the abutments is not new. References can be found in Danisy (Frézier, 1737-1739) in the 18^{th} century or Viollet-le-Duc (1854-68) in the 19^{th} century for instance. Quantitative analysis of those displacements is, on the other hand, a much newer subject. Some basic results were presented by Beranek (1989) but only on the influence of the displacements on the thrust in arches. Heyman in his classic papers on the analysis of masonry does not say much more (Heyman 1966). In the 1990s, experiments and a

first formulation of the problem was proposed by researchers of the University of Florence (Briccoli Bati, 1997). The more recent research were made by the author (Smars, 2000, 2010), Ochsendorf (2002, 2006) and Romano (2005). In his thesis, the author studied systematically the influence of large displacements on the stability of the arches and presented techniques to compute stability zones.
The results of the year 2000 were completed by more recent results gathered at the occasion of a research project. New experiments were done on the physical model of an arch (reproducing the proportion of the theoretical model used in the thesis). New software tools were also developed allowing great flexibility in the analysis.
In the following, a 2D static analysis is presented (negligible inertial forces). Each of the two abutments of an arch has three degrees of freedom (two translations and one rotation). Obviously, an identical translation of both abutments will not affect stability of an arch. On the other hand, an identical rotation will have an influence, as the direction of the forces on the arch will normally remain constant (in particular, if they are caused by gravity). There are therefore 4 degrees of freedom and the stability of an arch should be studied in this 4-dimensional space. In the following, a simpler situation will be analysed, where the abutments are not rotating. In that case, the system just has two degrees of freedom and a domain of stability can be defined in the space of the parameters u and v of relative translation of the two abutments.
The ability to study the stability of such free-standing arches is unlikely to be often directly useful for the structural analysis of real existing structures. Only barrel vaults may possibly benefit directly from the model. In our understanding, the treatment presented is more likely to bring understanding on the qualitative behaviour of arches and vaults than to produce accurate predictions.

2.1. *Physical model*

In 2008, a semicircular arch of 16 identical prismatic voussoirs was made and tested [**fig. 1**, Smars, 2010]. The arch has a span of 300 mm. Voussoirs are in stainless steel (to keep an accurate shape and resist the numerous experiments). Their bed joints have a square section of 30x30 mm^2. Friction between the beds is increased by covering them with a thin medical tape (~0.1mm).
An experimental rig is used to test the resistance of this and other arches

to large displacements. The left abutment of the arch is fixed. The right abutment can be displaced horizontally and vertically by two stepper motors. The rig is controlled by a computer which impose arbitrary deformation paths to the abutments. In a first series of experiments, the arches were deformed at a constant rate of 0.4mm/s.
The first results showed that some changes in the arch as it is deformed need time to occur. The experimental procedure changed accordingly: the deformation were imposed by steps of 1 mm separated by rest times (more details below).
A wooden centring is used to facilitate the construction of the arch. It is indeed important to keep the geometry of the arch as constant as possible. Even with such a provision, the models are never perfectly identical and the variations which are small after the arch is decentred (less than 1 mm) become important as it deforms.
A high-speed camera (Casio Exilim Pro Ex-F1) and a DSLR camera (Nikon D200) are used to monitor the deformation of the arch from the undeformed state up to collapse. The displacements are known with an accuracy of 1 mm (using calibration data from the motors and a few direct control measures).

2.2. *Deformation*

To accommodate large deformations of their abutments, material deformations are not sufficient; arches either form hinges between their blocks or blocks slide relatively to each other. In the present set of experiments, sliding was not observed and only hinges are considered.
A notation was devised to name patterns of hinges active as the arch deforms **[fig. 1]**. Blocks are numbered from 1 to 16, starting from the left side. Likewise, joints are numbered from 1 to 17. Hinges (revolute pairs) forming in between blocks are referred to by a positive number if the hinge forms at the extrados and by a negative number if they form at the intrados. ’-4’ indicates for instance an hinge forming in joint 4 (between block 3 and 4) on the intrados side. A deformed configuration normally involves more than one hinge. –4+9-14 is the hinge configuration forming when the horizontal distance between the abutments of the arch starts to increase. Deformed configurations form an isostatic structure which allows further deformation.
At some critical stage, a new hinge forms and the arch starts to collapse. –4+9–14(+17) denotes one of the possible collapse mechanisms.

The reference to the critical hinge is enclosed in between parentheses. At least four hinges are necessary to transform the three times hyperstatic arch in a mechanism (a planar quadrilateral linkage). When the arch collapses, some hinge further open but others may start to close. In the latter case, a prime ' is added to the sign associated with the hinge. (+1)-4+9+'17 indicates that, when it reaches his critical condition, the pattern –4+9+17 forms a new hinge at the extrados of the first joint +1 that hinge +17 starts to close. In any case, the angular velocities of a collapsing arch alternate: +–+–, –+–+, +–++, etc.

2.3. *Mathematical model*

A software program was developed in the framework of the PhD thesis to analyse arch structures: *Calipous*. It was recently rewritten to allow greater flexibility in analysis. In its present form, it is not a program anymore but rather a set of commands which can be used in a general-purpose scripting language (Tcl/Tk) to build, modify and query elements (blocks, joints, structures, points, forces, resistance criteria…). It can be operated interactively or from batch files. The key features of the models are the following:

- Structures are made of blocks (3D polyhedrons)
- Blocks are rigid and infinitely resistant
- Blocks have faces (2D polygons)
- Faces are rigid but may have a limited resistance (various criteria)
- A face is either in contact with the face of another block, or free
- External forces can be applied on free faces
- Calculations are made using the equations of equilibrium and criterion of resistance (not the deformability of the elements of the structure)

To analyse the influence of the deformation of its abutments, an arch with the same geometry as the physical model was built. The resistance criteria for the joints between the blocks imposes the forces to remain in the shape of the arch and to be compressive.
Algorithms were implemented in Tcl/Tk (with its new functions) to compute domains of stability of a hinge pattern (**fig. 2**, made with the “2000” version of the program) and to map the domains with values of the potential energy (**figg. 4-5**, made with the new version). A graphical user interface was

also written to interactively displace the abutments **[fig. 7]**. The position of individual blocks can be altered interactively (allowing the aperture of a new hinge, imposing some sliding, allowing a partial interpenetration of the blocks to simulate deformation…). Another interface controls the model and the experimental in parallel to facilitate real-time comparisons.
Figure 2 presents many of the theoretical domains of stability of the semi-circular arch investigated.

2.4. *Observations*

Arguably the most important experimental observation is that, for a given history of abutment displacements (for example: horizontal opening of the arch until collapse), results show significant variations (a situation also noticed by Romano, 2005). In some of the tests of the first series of experiments, the arch collapsed soon after it started to deform **[fig. 3]** and in others large displacements were necessary to reach the static limit. Small variations in geometry (global shape, the precision of the edges, initial sliding), material behaviour (joint) and abutments displacements (path, vibrations) have clearly the potential to lead to very different responses. Not surprisingly, this is always the case when considering the configuration of the blocks after their collapse. But actually, as the deformation proceeds, the variability of the results increases. In an attempt to reduce these effects, in a second series of experiments, greater care was given to the conditions of the test. To some extent, the variations decreased but they remained significant.

It is a common situation in science. The difficulty of long-term prediction of the weather is a classic example. In the framework of structural engineering, the rocking behaviour of block structures (like columns) also leads to large variations (DeJong, 2009).
For a set of experiments sharing similar initial conditions, more than one collapse mechanism is possible. This observation is directly related to the perfection of the model and to the extent to which parameters can be controlled during construction. In the first series of experiments, a few markedly different collapse mechanisms were observed. In the newer series, the collapse mechanism differ less, but the spread in the results is still important. This observation can be paralleled with the discussion made above on the domain of *contingent stability*: control and knowledge reduce uncertainty.
One of the reason behind the differences in responses is related to the

number of hinge patterns compatible with a given deformation and to the "distance" between them in terms of potential energy. For the same total displacement, there is often (but not always) more than one hinge pattern possible. **Figures 4** and **5** were prepared to compare the potential energy of a few hinge patterns: –4+9–14, +1+9–14 and +1+10–14. Figure 5 represents the sections in the surface along two histories of deformations. The energy required to pass from one configuration to the other is often low. The places where a bifurcation can occur depend very much on the initial condition. Because of a small difference at a critical moment, the structure may either continue its deformation following a path *a* or start following a new path *b*. This situation is similar to what occurs in buckling, another nonlinear structural phenomenon.

Another key observation is that the sections of arches between the hinges do not always remain rigid; sometimes joints start to open in those sections as well; sometimes four hinges of similar opening form and are stable **[fig. 6]** (an observation already made in 1732 by Danisy).
Three cases will now be discussed to present and possibly clarify the process at play.

2.5. *Horizontal deformation*

In this case, the arch is deformed by displacing horizontally its right abutment up to the collapse. This is probably the simplest case **[figg. 2, 7, 8]**.
When the arch starts to deform, three hinges form: two at the intrados of the arch (at the 4^{th} and 14^{th} joint) and one at the extrados (at the 9^{th} joint). This pattern is denoted –4+9–14. During the deformation of the arch, the line of resistance always passes through the three hinges. When it touches the shape of the arch at the extrados of joints 1 and 17, a new hinge forms, a mechanism is formed [–4+9–14(+17) or (+1)–4+9–14] and the arch collapses.
This is roughly what was observed in the experiments. A finer observation nevertheless shows that:
Some joints around the hinges open or deform slightly during the deformation. The eccentricity of the force in the joint obviously has an influence. But the effect is not reproducible. In some tests, the joints remained closed, in others, they open more significantly. One consequence is that the shape of the arch changes: usually increasing eccentricity and lowering safety **[fig. 7]**.

A few times, this slight opening clearly became a hinge: a fourth hinge (+10) which does not lead the structure to collapse immediately.
The behaviour is usually asymmetrical. Joints open preferably on the side of the moving abutment (hinge +10 rather than +8) **[fig. 7]**.
In the experiments, the arch collapsed significantly before the limit estimated by the model (32 mm). During the second series, the set of 20 experiments resulted in an average of 27.3 mm (standard deviation: 1.7 mm, minimum value: 25 mm, maximum value: 31 mm).
The model gives an upper-bound. Figure 7 shows that the shape of the arch is significantly different from what the model predicted.
The previous set of experiments showed even larger variations. In four cases, the collapse occurred in a slightly different manner: hinges –13 and –14 were both open during the deformation: three times the final pattern was –4+9–13(+17) and one time, the usual pattern 4+9–14(+17).
Figure 8 shows the domain of stability for –4+9–14 and –4+9–13. When the deformation starts, the pattern –4+9–13 is not admissible, only –4+9–14 is possible. But after a horizontal displacement of 15.6 mm, both patterns become possible. At this stage, the potential energy of –4+9–14 is 3731.6 mJ and the potential energy of –4+9–13 is slightly higher, 3733.4 mJ (1mJ is the energy that would be necessary to raise the full arch of 0.14mm). A small shock or a small difference in geometry can lead to a transition from –4+9–14 to –4+9–13. In that case, Figure 8 shows that the arch will collapse significantly sooner, after a total displacement of 26.6 mm instead of 32 mm.

2.6. Vertical deformation

In this case, the arch is deformed by raising its right abutment vertically up to final collapse. In the first set of 22 experiments, more than one collapse mechanism was observed. It was also discovered that some processes: opening and closing of hinges may take considerable time to occur (from seconds to minutes or even hours). As stated above, the experimental procedure was revised with the main intent to improve repeatability. The tests were conducted at a slower rate. For the first 40mm of displacement, the arch is displaced by steps of 1mm in 2s followed by 1s of rest. For this level of displacement, no slow-developing process was ever observed. In the second phase, the rest time is increased to 10s. This is not sufficient for all the changes to occur but enough for most of them. As the process is automatically controlled, the history of deformation is the same for all the tests, facilitating comparisons.

Like in the case of the arch opening, fewer variations were observed in the new set of 20 experiments. Some observations are identical. The joints around the hinges open slightly, especially around –14 and +9. The geometry is therefore not what the model predicts, especially when the displacements are large, and the graphs have to be interpreted with caution.
During the second series of experiments, the set of 20 experiments resulted in an average of 51.2 mm (standard deviation: 2.5 mm, minimum value: 47 mm, maximum value: 56 mm). It never approached the theoretical limit.
As the abutments move, the line of resistance approaches the extrados at the level of joint 10. At around 40 mm, in most experiments, both hinges +9 and +10 are clearly open **[fig. 6]**. As the arch further deforms, hinge +9 closes and hinge +10 opens: sometimes fast, more usually slowly. The full sequence is documented in figure 10. The arch usually collapses before hinge +9 completely closes. A fast transition always leads to a direct collapse. In an intermediate position, block 9 is only marginally stable. Tests showed that in this position, the arch left unattended may collapse after a few minutes or still be standing after one hour **[fig. 6]**.
At a certain level of deformation (after about 50 mm), three critical situations were met as the line of resistance approached hinge –5, +10 and +17. In the mathematical model, the arch can continue its deformation. In the experiments it cannot, the shape not being exactly the same. When the line touches –5 (the most common occurrence) or +17, it collapses.
The situation with hinges +9 and +10 both open is particularly interesting. The line of resistance always needs to stay inside the shape of the arch. At some stage, it is possible for both +1+9–14 and +1+10–14. This is also the case for one intermediate configuration. When hinges +9 and +10 are both open, with a proportion depending on the vertical deformation of the abutment, the line of resistance is parallel to the extrados of block 9 and passes by the two hinges +9 and +10, i.e. it stays inside the shape of the arch. That does not imply that this configuration (in equilibrium and admissible) is stable. To understand better this case, the potential energy of the arch was computed in function of the vertical displacement of the abutment for patterns +1+9–14 and +1+10–14 and all the intermediate configurations **[fig. 12]**. As figure 5-top was already showing, for a vertical displacement of 40 mm, the potential energy of +1+9–14 is the lowest but, for a displacement of 50 mm, this is +1+10–14. The new figure also shows that intermediate configurations have always a higher potential: there is

always a barrier of potential to overcome to pass from one configuration to the other. The "watershed" actually corresponds to the situation where the line of resistance passes by the hinges +9 and +10, a situation therefore associated with an unstable equilibrium. Nevertheless, this is the situation which is sometimes observed. In our understanding, that can only mean that the potential energy of the external forces alone cannot be used to study the stability of the configuration. The small model arch is very rigid but it is nevertheless necessary to add the deformation energy in the joints and consider the total potential energy. When +9 and +10 are both open, the forces on block 9 are its weight and two internal forces passing by the hinges. Forces passing by a perfect hinge do not offer any resistance to the rotation of the block. The fact that the block settles down in this configuration indicates that the hinges are not perfect and offer a (very small) resistance to rotation. The experiments show that this equilibrium is very unstable and can quickly lead to a collapse.
In the first series of experiments, a greater variety of failure mechanisms was observed. Two times, the arch collapsed by forming an out-of-plane mechanism **[fig. 3]**. At some stage (unpredictable), the three hinges become three spherical pairs, the arch gains six more degrees of freedom, becomes a mechanism and collapses. This situation results from initial slight misalignments between the blocks framing a future hinge (blocks 8 and 9 for instance). In this case, potential hinge lines would lead to out-of-plane displacements incompatible with the movements imposed on the abutments. Obviously, small rigid models with thin joints are more sensitive to this kind of failure than large arches. This kind of failure exemplifies a common situation: events do not always conform to the rules chosen by modellers.

2.7. *Vertical deformation (after a horizontal deformation)*

In this case, the right abutment of the arch is first moved 10mm to the right (opening) and then deformed vertically up to the collapse.
There are little variations, especially at the beginning of the experiments. The hinge at –13 progressively closes and, after a displacement of about 15mm, when it is completely closed, hinge +17 starts to open. As in the other experiments, the joints around the hinges are most often slightly deformed or open, in a proportion which changes from experiment to experiment, resulting in unforeseen variations of the geometry. In particular, joint +10 usually shows a slight opening.

In some cases, this opening was already large when the vertical movements started: a real hinge. The last phase of the experiment is again often characterised by a transition between hinge +9 and +10. The transition does not necessarily occur and when it occurs, it is not always at the same time and rate. In earlier phases, no clear indicators are pointing towards a particular evolution.
Again, the spread of the results is important. The set of 20 experiments resulted in an average of 55.3 mm (standard deviation: 3.9 mm, minimum value: 47 mm, maximum value: 61 mm).
In the previous series of experiments, more collapse mechanisms were observed: four in the 20 experiments. The arch collapsed 8 times by an out-of-plane mechanism **[fig. 3]**. The critical hinge sometimes occurred in +17 and sometimes in –13 or –14. In the final stage, the mathematical model shows that just before collapse, the line of resistance passes very close to these points.

3. *Conclusions*

The model of arches subjected to large displacements of the abutments presented above is not particularly effective. In its present form, it certainly fails to provide robust safety estimates. Even in laboratory conditions, variations are important. The behaviour of the joints was found to be critical in that respect. Arguably, it cannot be predicted deterministically. This was even the case here, with very thin joints having a zero (or very low) resistance to tension. The spread also depends strongly on the quality of the construction. These observations prompt the broader approach advocated at the beginning of the thesis. Here specifically, of integration of mathematical models and direct measurement of the deformations (and of the pathology). This fact is acknowledged but should now be implemented.
From a qualitative point of view, the results are more positive. A few mechanisms were identified and the model helped to understand them.
Inaccurate estimates are not uncommon. Historical structures are always imperfectly known. In this context, believing in predictability requires a lot of faith. Ideally, it should be possible to give a probabilistic measure of safety based on a probabilistic knowledge of the situation. This is an interesting but hard road to follow (Smars, 2012). Most often, the required data is not available. Sometimes, because of the uniqueness of the situa-

tions investigated, it may not be possible to gather it. Limit analysis takes a middle road. It requires a measured level of faith but demands less from the models. This is along this course that the thesis fared.
Some consequences of the occasional failure of the most common hypotheses of limit analysis were discussed and partially investigated. At this stage of our understanding, the result is often an unwelcome complexity. To some extent, it is probably unavoidable. But, arguably, it may contribute to a better understanding of the limits of our models.

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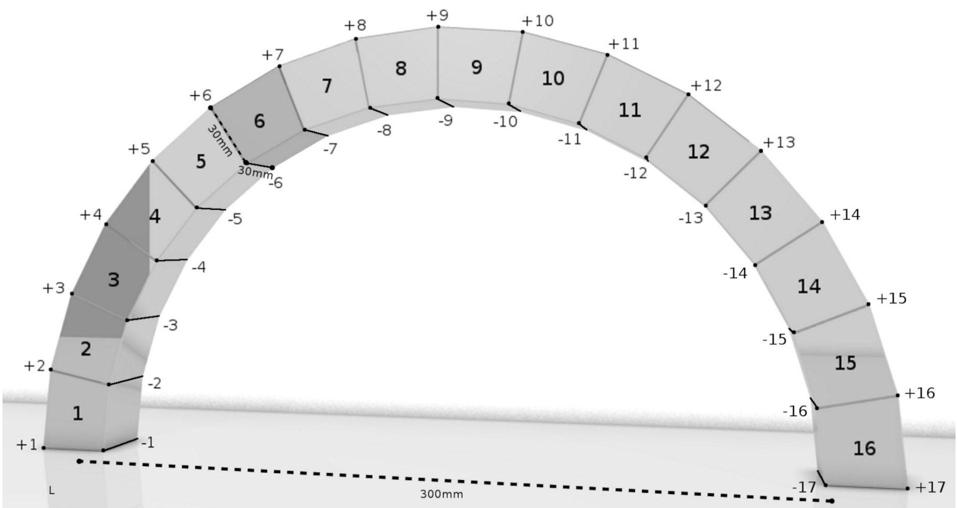

Fig. 1. Model of a semi-circular arch made of 16 identical voussoirs. Blocks are identified by a number and potential hinges by a signed number: positive on the extrados and negative on the intrados.

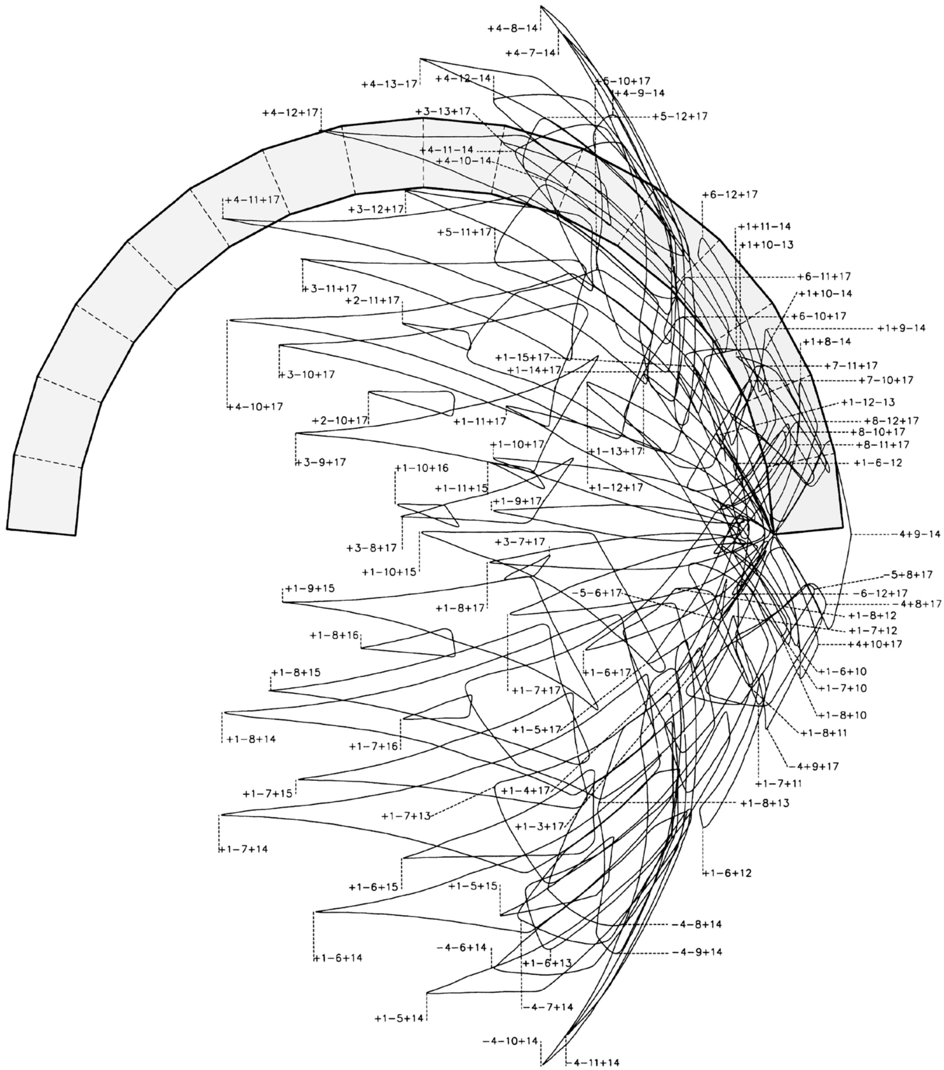

Fig. 2. Kinematic Domains of Stability of the hinge patterns of a semi-circular arch.

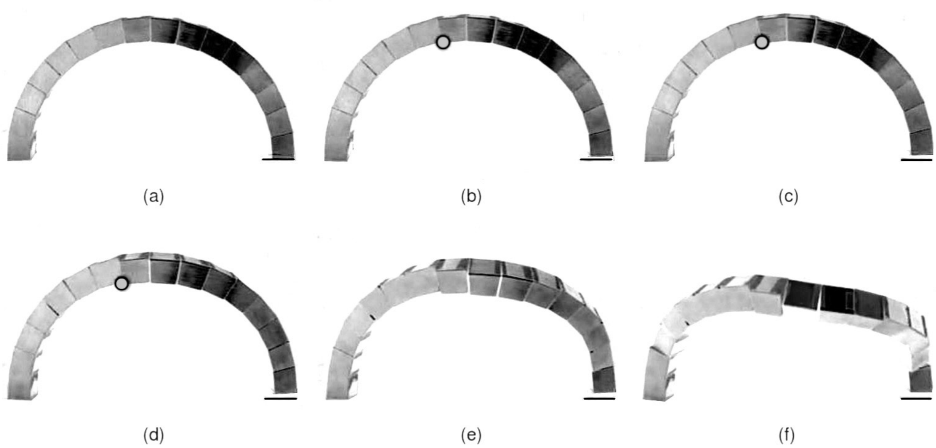

Fig. 3. Relative vertical displacement of the abutments of a semi-circular. Out-of-plane collapse.

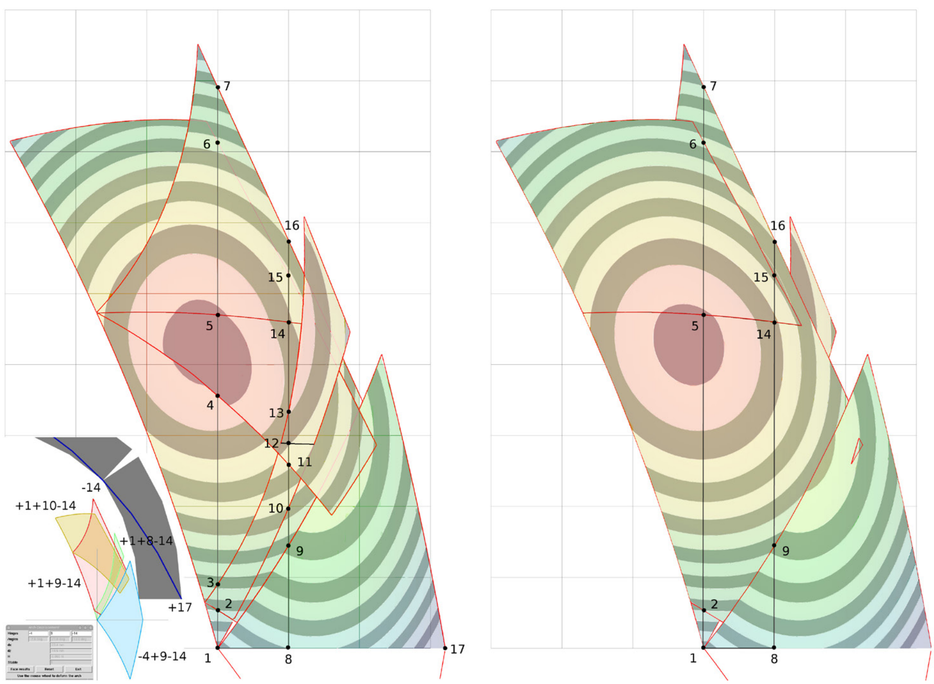

Fig. 4. Potential energy associated with three hinge patterns (only the difference of level between the sheets and at a specific point are meaningful). Left: potential surface seen from the top (high energy). Right: potential surface seen from the bottom.

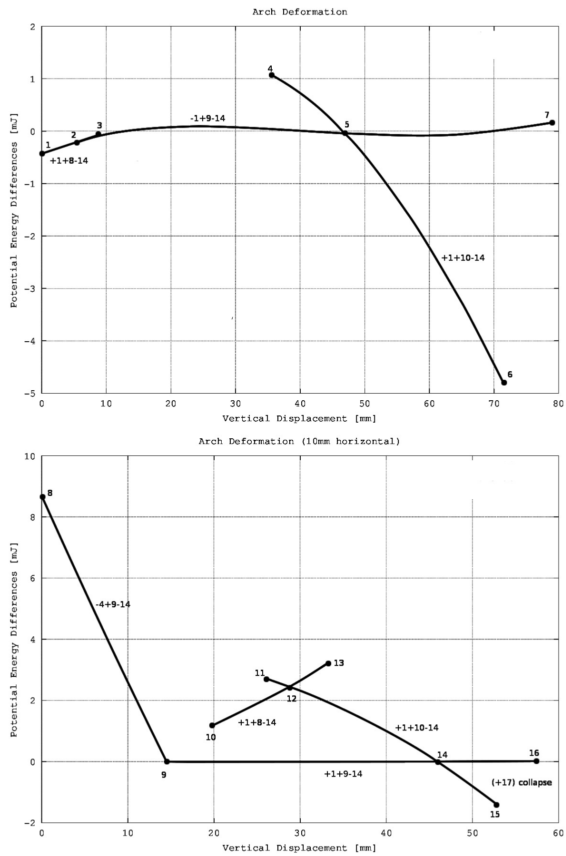

Fig. 5. Potential energy differences of three hinge patterns as the arch deforms (again only the distance between the lines is meaningful). Top: vertical displacement of one abutment, up to collapse. Bottom: horizontal displacement of 10 mm followed by a vertical displacement up to collapse. (the critical points are also marked on Figure 4).

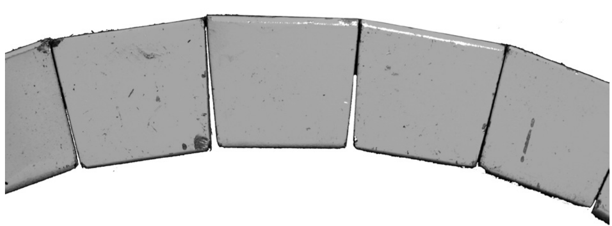

Fig. 6. Deformation of the semi-circular arch. Detail showing two hinges forming in +9 and +10 for a displacement of 45 mm. The arch remained in that configuration for one hour, before the displacements were increased bringing it to collapse.

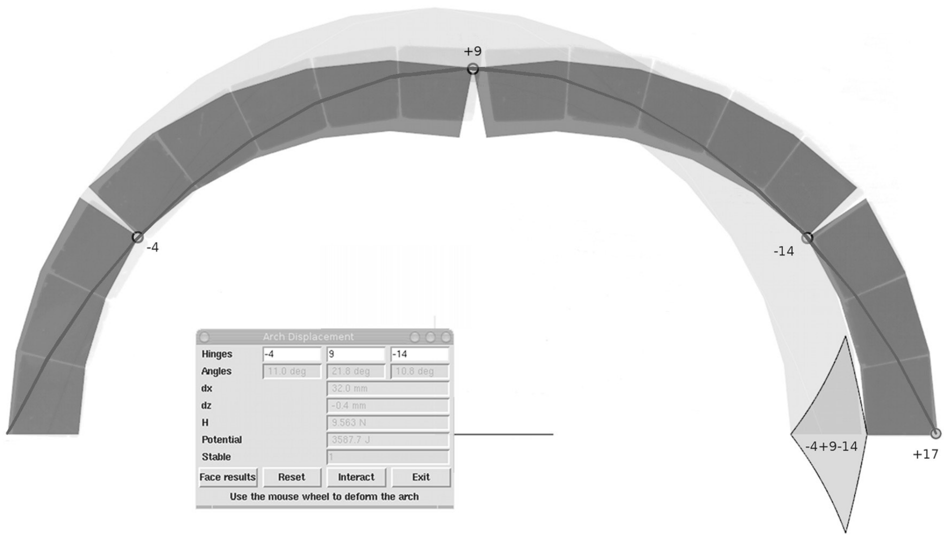

Fig. 7. Computer analysis of the mathematical Model. Arch opening. Configuration just before collapse (horizontal displacement: 32 mm). The image of a real experiment (the biggest opening observed) is superimposed on the model.

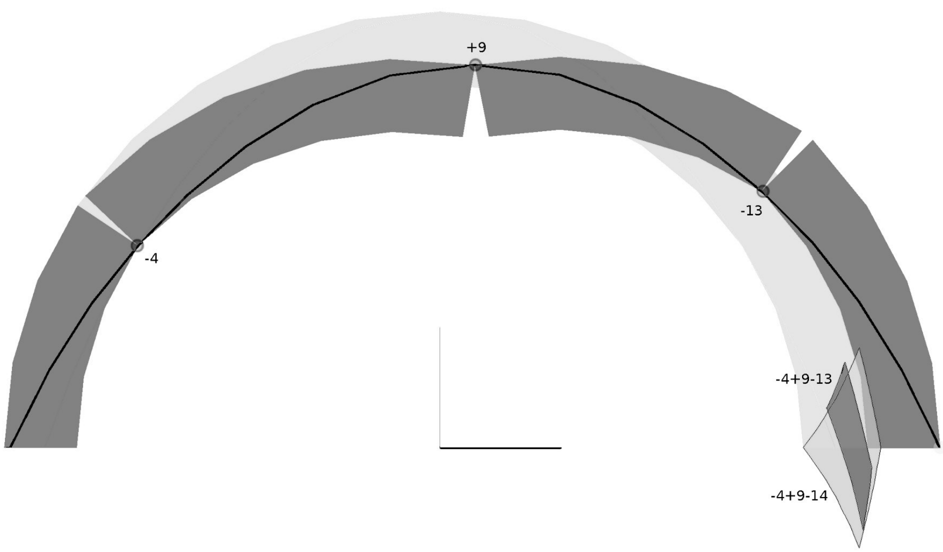

Fig. 8. Horizontal displacement, patterns 4+9–14 and 4+9–13 leading to a faster collapse of the arch, after 26.6 mm.

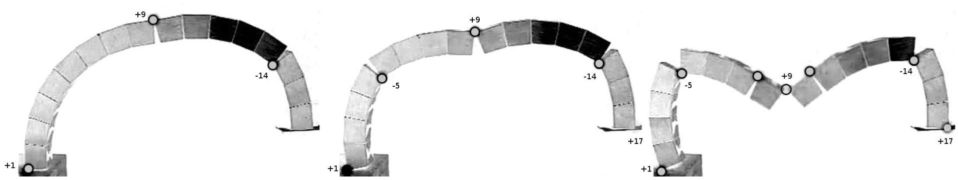

Fig. 9. Vertical displacement. Left: formation of three hinges to allow deformation. Middle: a fourth hinge forms at –5 and the arch collapses; just after a hinge forms in +17. Right: more hinges forms, some blocks loose contact.

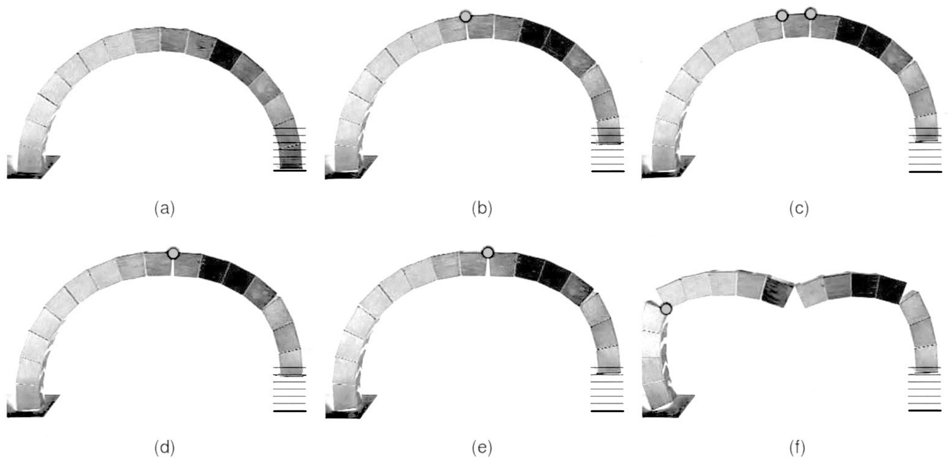

Fig. 10. Vertical displacement. Evolution of the hinge patterns. a: undeformed arch, b: formation of hinges at +1, +9 and –14, c: a new hinge starts to open at +10, d: hinges open at +9 and +10 (see also figure 6, e: hinge +9 closed, f: a new hinge forms at –5 and the arch collapses.

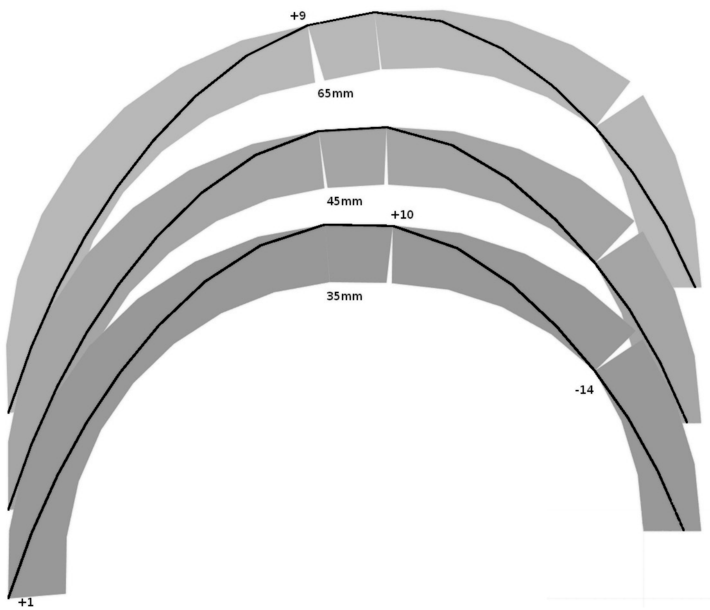

Fig. 11. Three configurations of the arch where the line of resistance is parallel to the extrados of voussoir 9 and passes by hinges +9 and +10.

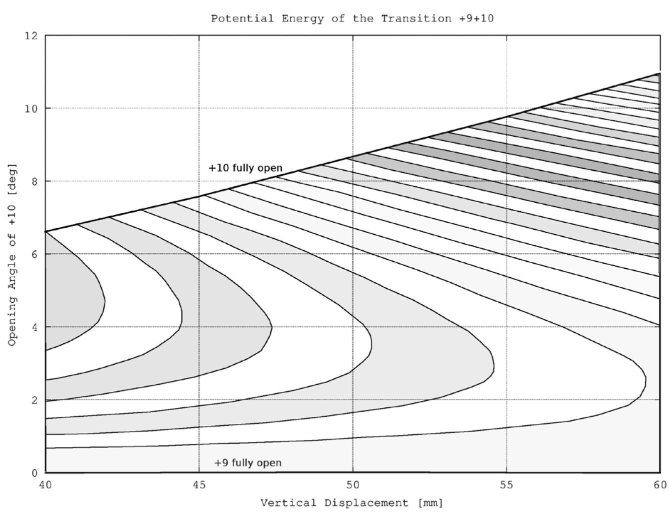

Fig. 12. Potential energy of the arch corresponding to the hinge pattern –4+9+10–14. This pattern has one degree of freedom and can lead to a spectrum of configurations. The potential energy is computed in function of the vertical displacement of the abutment and in function of the opening angle of hinge +10.

Ilaria Pecoraro

I sistemi voltati del Salento: origini, geometrie costruttive e problemi di conservazione*

1. *Origini e matrici d'ispirazione formale*

Il presente contributo riassume anni di studio e di cantiere di restauro delle 'volte leccesi' indagate sotto l'aspetto storico-critico e tecnico-costruttivo (Pecoraro, 2000, 2004a, 2004b, 2005a, 2005b, 200c, 2005d)[1].
La ricerca evidenzia il confinamento di questo fenomeno di lunga durata nella provincia di Terra d'Otranto (Leccisi, 2003; Galante, 2006; Fallacara, 2012)[2] e lo interpreta, a primo acchito, come la rimodulazione periferica, specialistica e standardizzata dell'esperienza aulica di ambiente valenziano nella seconda metà del Quattrocento, ad opera del '*mestre*' Francesc Baldomàr e dei suoi allievi (Catalan, 1997; Pecoraro, 2003)[3].

* Nato in seno al corso di Dottorato di Ricerca in Restauro dei beni architettonici (XIII ciclo, anni 1999-2002), presso la facoltà di Architettura di Roma, Università 'La Sapienza', il lavoro monografico ha ricevuto una menzione speciale in occasione del Primo Premio in onore di Edoardo Benvenuto nel dicembre 2002. Ringrazio la Fondazione Franzoni per aver concesso l'opportunità di condividere lo stato dell'arte del tema di ricerca di una vita.

[1] L'argomento è taciuto fino alla fine dell'Ottocento nei documenti di archivio scrittografici consultati e poco studiato anche in ambito applicativo-professionale. Ancora oggi, a distanza di venti anni dalla discussione della tesi di Dottorato, pur avendo l'autrice collaborato alla promozione del testo di legge regionale n. 26 del 27/10/2009, *"Tutela e valorizzazione del sistema costruttivo con copertura a volta"* e portato avanti campagne di sensibilizzazione sui temi della conservazione dei sistemi voltati tradizionali in pietra della Puglia (presso gli ordini professionali, le C.L.P. comunali e le pubbliche amministrazioni), si rileva nel merito degli interventi una diffusa ignoranza culturale e la tendenza alla 'sostituzione' integrale di queste volte con solai misti di ultima generazione, se interessate da significativi quadri fessurativi. Infatti, la conoscenza dei relativi caratteri costruttivi, propedeutica per un corretto intervento di recupero o di restauro critico-conservativo, e i relativi aspetti di compatibilità materiale e statica degli interventi sulla preesistenza, sono disattesi nei capitolati di appalto e nei prezzari regionali ufficiali.

[2] La Terra d'Otranto comprendeva le attuali province di Brindisi, Lecce e Taranto e quota parte dei territori di Matera e di Monopoli.

[3] Francesc Baldomàr è il *protomagister* nel cantiere del sistema voltato in folio, ad ipografia stellare composita, in pietra vulcanica, edificato sotto il governo dei sovrani Isabella di

Il confronto tipologico, tecnologico e formale della volta stellare valenziana con quelle salentine mette in evidenza alcune similitudini nella logica costruttiva a filari isometrici 'in folio' a calotta autoportante e nel disegno delle ipografie 'a spigolo e a squadro'. L'impresa edilizia castigliana è innovativa per materiale, forma, taglio stereometrico dei conci; probabilmente la sua arditezza costruttiva desta già all'epoca molto interesse, richiamando l'attenzione di colleghi locali e no.
Vero è anche, però, che mentre in Valencia l'esperienza "stellare" resta episodio importante ma isolato, ricorrente anche in ambito francese (e questo lascia aperti ulteriori filoni d'indagine e interpretativi), (De L'Horme, 1561; Vandelvira, 1575-1591)[4] nel Salento è plausibile ipotizzare che le volte stellari diventano, ininterrottamente a partire dalla seconda metà del Cinquecento in poi, il principale e poi l'unico metodo costruttivo di orizzontamento. Questo aspetto le rende fattore identitario della storia dell'architettura salentina in Età Moderna, degno di conoscenza, tutela e conservazione.
L'indagine storico-critica non si esaurisce nell'individuazione del prototipo castigliano, che da solo non giustifica le varianti e i caratteri che connotano il caso di studio. In un territorio stretto fra due mari, facile preda dei corsari saraceni, costantemente conteso fra veneziani, francesi, spagnoli, musulmani, oltre che dalla Chiesa di Roma che avvia il processo di latinizzazione dei territori, insediandovi nuovi conventi di Teatini e di Domenicani, il fenomeno subisce molteplici influenze che ne orientano affermazione, diffusione e lunga durata **[fig. 1]**.
Le volte stellari fanno la loro comparsa in area neretina e leccese nell'arco temporale che va dal martirio di Otranto (1481) alla battaglia di Lepanto (1571). Sono precedute da forme semplificate di volte a botte lunettate lungo le direttrici e nelle soluzioni d'angolo (come ad esempio nel castello di Acaya in Lecce e nella chiesa di San Pietro in Ottava presso Montalbano di Fasano, lungo la via Appia Traiana) **[fig. 2]**.

Castiglia e Alfonso d'Aragona nel convento domenicano di Valencia (1439-1467). Si ringrazia l'arch. Federico Iborra per avermi introdotto nel 2000 allo studio del sistema voltato di Valencia e l'arch. Arturo Zaragozà Catalàn per aver pubblicato la ricerca in Valencia.

[4] Il taglio stereometrico dei conci accompagna gli organismi architettonici francesi dal Medioevo all'Età Moderna, con impiego di una pietra spesso simile a quella leccese, soprattutto in area parigina.

La tecnica costruttiva stellare raggiunge complessi livelli formali nella seconda metà del XVIII secolo, con soluzioni sempre più articolate: nel disegno ipografico; nella cura dei piani di appoggio fra ‘punte’ e ‘*appese*’; nella sapiente apparecchiatura di diatoni intrecciati fra loro con giunti sfalsati; negli apparati decorativi di pietra; nei conci di chiave monoblocco, in pietra “leccese” o “gentile”, serrati nel baricentro delle calotte con funzione di diatono, interconnessi alle ‘linee’ dei filari che compongono le strutture portate in *folio* (Fonseca, 1979; Manieri Elia, 1955, 1956, 1979, 1990; Brandi, 1967, 1991; Cazzato, 1989, 1987; Nobile, 2016)[5].
Un ulteriore impulso sperimentale viene fornito dalla vivace attività dei cantieri di ricostruzione post-sisma del 20 febbraio 1743 **[fig. 3]**[6].
Nella seconda metà del Settecento, e soprattutto a seguito del terremoto calabrese del 1783, si avviano sperimentazioni di cantiere protese alla realizzazione di elementi di presidio antisismico (*pontoni*, *homo morto ligneo*, catene metalliche, contrafforti murari esterni, ispessimento e incamiciamento delle sezioni murarie a tutta altezza). Si realizzano perfino nuovi sistemi voltati stellari, non più in pietra ma in incannucciato intonacato e stuccato (dello spessore di 3, 4 cm) a forma stellare composita, sostenuti da un graticcio che simula le carene delle navi in estradosso, con ancoraggio ligneo alle coperture a doppia falda o a capriata (come ad esempio verificato di persona nella chiesa settecentesca della Natività della Vergine detta chiesa Nova in Lecce).
La sostituzione di coperture lignee fatiscenti e imbibite di acqua con sistemi di orizzontamento e copertura in pietra, voltate a stella, determina un’incisiva rivisitazione anche della geometria, del dimensionamento e della distribuzione dei sistemi portanti verticali.
Dalla seconda metà del Settecento alla prima metà dell’Ottocento si registra un ulteriore fenomeno tecnico-costruttivo finalizzato a realizzare strutture murarie portanti irrigidite da speroni murari che sostengono le spinte dei sistemi voltati a stella. Vi si introducono cunicoli-contrafforti che esercitano la funzione di pseudo-archi, all'interno di corridoi passanti, coperti da triangoli ‘di scarico’. Questa scelta progettuale è ben evidente

[5] Si approfondiscano i temi oggetto di analisi in questi studi.

[6] Il sisma del 20 febbraio 1743, detto di Nardò, di magnitudo stimata M 7.1, causa ingenti danni, crolli, vittime (<https://ingvterremoti.com/2020/08/17/la-sismicita-storica-del-salento-il-forteterremoto-del-20-febbraio-del-1743/>).

nei sistemi voltati stellari compositi, detti anche 'ad ombrello', 'a trulla' o 'cappelloni', impiantati su preesistenti sezioni murarie, opportunamente incamiciate e tirantate da travi lignee annegate nelle stesse sezioni murarie portanti[7].
Post sisma del 1743 le città salentine cambiano volto. Gli skyline dei centri storici *intra moenia* vengono ridisegnati, passando da soluzioni a doppia falda in legno in edifici ad un piano, a soluzioni a due e più livelli, con coperture "a trulla" rifinite con coppi estradossali o con 'lastrico' a base di cocciopesto e calce **[fig. 4]**.
L'Ottocento rappresenta il secolo della diffusione capillare di questo metodo costruttivo oramai 'standardizzato'. In virtù del suo carattere 'prefabbricato', viene utilizzato in ogni tipologia edilizia (religiosa e civile, residenziale e lavorativa, di rappresentanza ecc). Negli anni ottanta del XIX secolo compaiono infine i primi testi di Architettura tecnica (De Giorgi, 1879; Gentile, 1856, 1878; Arditi 1878, 1888, 1894)[8] che computano lo sviluppo lineare delle superfici d'intradosso delle volte 'a stella e a squadro'. In questi testi non si analizza né la genesi storico-critica, né i caratteri tecnico-costruttivi e statico-dimensionali, ma si cominciano a disegnare in pianta le proiezioni stellate. Il relativo *modus erigendi* costituisce oramai in Terra d'Otranto il bagaglio di conoscenze trasmesse verbalmente e con l'esperienza di cantiere di padre in figlio nelle squadre dei 'voltaroli'[9].
Nella prima metà del Novecento la tecnica costruttiva trova ovunque campo di applicazione, essendo utilizzata anche nella costruzione di elaiopoli, frantoi fuori terra, tabacchifici, stalle, cantine di produzione di vino, luoghi della trasformazione dei prodotti agrari, masserie ecc, assecondando dimensioni e procedure codificate (Baffa, 1954).
L'avvento dei solai in calcestruzzo armato gettato in opera (solai del tipo 'a margherita' degli anni 1920-1965) e dei solai misti in laterocemento (dal 1965 ad oggi) soppianta integralmente i sistemi voltati stellari.

[7] La soluzione 'a trulla' viene disegnata nei prospetti di fine Ottocento del Palazzo della Corte in Ostuni; essa è oggi rilevabile nel Convento delle benedettine *intra moenia* nel rione Terra di Ostuni e nei casi di studio del cappellone ad ombrello all'ingresso della chiesa madre di Carovigno (XVIII secolo) e dei cappelloni del Santissimo e di san Gregorio Magno nella Collegiata di Manduria (rispettivamente prima e seconda metà del Settecento).

[8] Si rimanda per opportuni approfondimenti alla consultazione della corposa documentazione bibliografica citata.

[9] Questa pratica accomuna la tecnica costruttiva 'a stella' alla tecnica costruttiva 'a trullo'.

Dal 1990 al 2010 si registrano timidi tentativi di studio e di riproposizione della soluzione stellare nei cantieri edili in cui, però, le volte a stella di ultima generazione sono snaturate sia nella forma del sesto (spesso ribassato) sia nella logica distributiva delle forze peso, convogliate sui celati telai in cemento armato; in casi estremi 'le stelle' sono pseudo-volte in polistirolo, una sorta di controsoffittatura dalla pseudo forma 'a spigolo o a squadro'.

2. *Costruzioni geometriche*

La costruzione geometrica si fonda sull'assemblaggio incatenato di superfici a differente curvatura, impostate su quattro archi a due a due paralleli e perpendicolari fra loro (sì da generare una cerchiatura in pietra). Si realizza un sistema voltato autoportante, omogeneo, pseudo-isotropo, composto da elementi finiti (i conci) fra di loro incastrati quasi a secco e disposti di piatto (in folio).
La costruzione geometrica delle volte leccesi è interessante in quanto contraddistinta da fattori standardizzanti la fornitura e la lavorazione a piè d'opera e in corso d'opera degli elementi che le compongo, ovvero:

- piedritti o murature continue di appoggio;
- *appese* e *cunucchiedde*;
- *punte* stellari, dette anche *fusi*;
- *calotta*;
- concio di chiave;
- *mortiere*;
- *battuto* di *cocciopesto* in estradosso.

Dal punto di vista tipologico le volte 'leccesi' si chiamano:

a. 'volte a spigolo o a stella' se a quattro punte;
b. 'volte a squadro aperto' se ad otto punte supportate da squadri aperti a 90°;
c. 'volte a squadro chiuso', se ad otto punte con squadri disposti a 45° rispetto ai due muri perpendicolari fra loro, a mò di mensola dell'unghia sovrastante;
d. ' volte stellari' composite, se a 12, 16 punte;
e. 'volte ad ombrello', anche con soluzione di pseudo-tiburio ('a *trulla*') se composte da fusi fra di loro privi di nervature costolonate, con conci serrati ad incastro fra loro **[fig. 5]**.

Gli ambienti voltati hanno impianti dalla forma quadrangolare, rettangolare, ottagonale regolare, circolare e si costruiscono impostando sui quattro piedritti angolari una specie di pennacchio sferico su cui ordire gli archi (solitamente a sesto acuto, voltati 'alla maniera di Santa Caterina di Galatina' (Testamento Caballerio, 1558; Bianco, 2016)[10]. Gli archi supportano le *appese* fino alla terza linea di 'giro' (posto a circa 30°-32°). Sull'*appesa* s'innesta l'unghia, costruita contestualmente su quattro cantonali senza ausilio di centinature ma con l'aiuto di una 'forma', un arco di legno inchiodato in cantiere che segna la curvatura del sesto. Terminate e cerchiate le 'unghie', si ordisce per quadrature concentriche ogni filare componente la calotta superiore, posta sovrasquadro di circa 3-4 cm rispetto il piano delle unghie, per garantire la serratura dei conci fra loro; il concio di chiave chiude infine la calotta.
La peculiarità formale e geometrica di queste volte è insita nell'assemblaggio di superfici sinclastiche a differente curvatura, poste tra di loro con la logica del mutuo contrasto strutturale.
Il *mortiere* (riempimento sciolto misto di terra, schegge lapidee, cocci pestati, conchiglie, tufina, filamenti vegetali e residui della lavorazione dei conci, calce aerea allo stato semigrasso, battuto e ribattuto per sottocantieri con le tavole legate ai piedi) funge da 'carica' inerme, al fine di ridurre la componente orizzontale delle controspinte generate dai mutui contrasti delle porzioni di superfici sinclastiche a curvature variabili.
Scaricare il sistema voltato del suo mortiere equivale ad attivare l'esplosione verso l'alto dei conci apparecchiati in folio.
Osservati dal basso i sistemi voltati stellari manifestano due peculiarità estetico-strutturali:

a. una forma geometrica simmetrica lungo i due assi baricentrici, ortogonali fra loro;
b. l'assenza di nervature portanti di scarico, di costoloni irrigidenti e costituenti lo scheletro portante delle parti tamponate, nei sistemi voltati a crociera quadrati, pentapartiti o esapartiti, propri dell'architettura gotica.

[10] Enza Aurisicchio, Direttore dell'archivio diocesano di Ostuni ha trovato, trascritto e studiato la copia del testamento di Caballerio (1423), in cui il committente chiede che alla sua morte sia costruita una cappella in onore del Santo di Compostela, coperta con sistema voltato a botte a sesto acuto, alla maniera delle navate della Basilica di Santa Caterina d'Alessandria in Galatina.

In tal senso, quindi, il comportamento strutturale dei sistemi voltati leccesi non è assimilabile a quello di una volta a crociera costolonata, diffusa negli organismi architettonici salentini normanni, angioini e orsiniani.
Al termine del suo montaggio, la volta a stella si comporta come una calotta monolitica autoportante e omogenea, con spessore costante di sezione reagente (altezza del concio di 15-18 cm ca), aggregata da incastri generati dalla giuntatura sfalsata delle linee, le une rispetto le altre, e poggiata sui piloni quadrangolari portanti e laterali, uniti insieme da archi di 'irrigidimento'.
Rimandando alla consultazione dei contributi già pubblicati, preme in questo contesto sottolineare come la forma dei sistemi voltati leccesi asseconda comportamenti strutturali efficaci a fronte di un impiego di minore materiale da costruzione e una pre-progettazione e pre-dimensionamento e taglio dei conci ai piedi dell'opera stessa. Infatti, impostata la struttura portante verticale risulta 'meccanico' per il maestro di muro, assemblare i pezzi notevoli fra loro, seguendo una sequenza normata, sperimentata dalla pratica di cantiere, quindi affidabile.

3. *Problemi di conservazione*

La ricerca, affiancata da esperienze dirette di cantiere di restauro statico ha messo a punto e poi verificato dal vivo la bontà delle proposte avanzate dalle scelte di restauro tradizionalmente compiuto sui sistemi voltati stellari, con indicazioni d'intervento fondate sul rispetto dei principi di reversibilità, distinguibilità, compatibilità fisico-chimica, minimo intervento sulla preesistenza (Bonelli, 1963; Carbonara, 1992, 2000; Palmerio, 1996).
Gli aspetti progettuali del problema della conservazione e quelli creativi dell'intervento di manutenzione e di restauro sono affrontati all'interno di una maglia vincolante di fattori storici, estetico-formali, tecnici e tecnologici, statici e socio-culturali, in linea con il criterio di "unità teoretica e metodologica" della disciplina del restauro (Carbonara, 1997).
L'obiettivo dell'intervento è quello della conservazione critica del valore storico ed estetico della volta salentina, evitando impossibili ritorni "alle origini", "mode" di restauro prive di alcun fondamento storico-critico e operazioni tese alla "pura" conservazione o al ripristino.
Innanzitutto non è possibile scindere la fase di restauro statico del sistema voltato stellare salentino dalla preventiva valutazione del livello di conservazione e di efficienza statica dei relativi sistemi verticali portanti.
Nel primo come nel secondo elemento si deve preliminarmente valutare

lo stato di conservazione dei sistemi portanti e portati in muratura a conci squadrati, a filari isometrici (verticali e orizzontali), attraverso indagini dirette e indirette finalizzate a stendere un quadro conoscitivo su eventuali meccanismi di collasso in corso. In questo ci aiuta il rilievo dei quadri fessurativi (superficiali, profondi e passanti).
Ogni struttura voltata soffre di patologie strutturali indirettamente o direttamente apportate alle sezioni reagenti: indirettamente allorquando i piedritti o gli elementi murari verticali di sostegno vengono interessati da cinematismi di rotazione e traslazione, nel piano e fuori dal piano; direttamente, allorquando è la sezione muraria orizzontale componente la volta stellare a manifestare degrado dei materiali e deperimento delle strutture causato quasi sempre da acqua da infiltrazione, generati o da mancata manutenzione o da danni antropici apportati alla struttura da improprie azioni 'edilizie'.
La maggior parte degli interventi qui proposti tendono a limitare il danno generato da cause che possono essere facilmente arginate con opportuni non invasivi interventi di restauro statico. Si provvede a migliorare, se necessario, la capacità portante della struttura voltata, a diminuire la spinta degli archi sui piedritti e a preservarne lo stato di salute, anche mediante un'attenta e appropriata programmazione della manutenzione.
La frequentazione diretta dei cantieri salentini odierni, in cui siano ancora adottati alcuni metodi costruttivi tradizionali e la conoscenza più approfondita dei sistemi voltati e murari a filari isodomi della Terra d'Otranto consentono di formulare importanti riflessioni sul tema.
Gli interventi più corretti sulla preesistenza, soprattutto nel caso d'edilizia di base, sono contraddistinti dal loro intrinseco carattere minimalista, discreto e prudente, economicamente vantaggioso e di semplice realizzazione, sia nella tecnica applicativa sia nel reperimento di materiali locali[11].
Salvo casi particolari, le operazioni che impropriamente vengono compiute sulle strutture salentine (cordolature in cemento armato, stonacatura,

[11] Salvo casi delicati di restauro, la semplicità degli interventi proposti in questa sede scaturisce dal carattere omogeneo del materiale costruttivo impiegato ovunque; dalla regola costruttivo-tecnologica che accomuna la costruzione del muro a quella della volta, del terrazzo, del pavimento, dell'elemento di finitura; dall'invarianza della dimensione delle *taglie* di cava, che coltivano conci della stessa dimensione di quelli estratti cinquecento anni addietro, nelle province di Terra d'Otranto.

impiego di malte cementizie ecc), non solo non restaurano la materia, ma alterano l'autenticità strutturale degli elementi, modificano l'aspetto dell'opera d'arte e spesso accelerano il processo di degrado della materia lapidea e di deperimento delle strutture, oltre a far lievitare i costi complessivi dell'intervento (Giuffrè, 1997).

L'intervento di reintegrazione, compiuto mediante la sostituzione d'interi conci viene dettato dalla natura stessa della materia, ma deve essere applicato con moderazione tentando, ove possibile, di garantire "l'attualità espressiva" e di consolidare con metodi reversibili i conci originari delle murature, specialmente quando questi sono connotati da una notevole valenza storico-estetica (Palmerio, 1996).

La conoscenza delle tecniche costruttive tradizionali diviene, pertanto, un'esigenza attuale improrogabile, che non deve però tradursi in necessità-volontà di compiere interventi di ripristino analogico di forme del passato con tecniche del pseudo passato. Gli antichi mestieri, infatti, non possono essere riesumati. Si deve invece tentare un'operazione di dialogo e d'integrazione fra conoscenze della tradizione costruttiva locale e conoscenze offerte dagli attuali orientamenti di quella branca della disciplina del Restauro che si occupa di analisi del comportamento statico degli originari organismi architettonici, sviluppando proposte di intervento compatibili con le logiche strutturali insite nello stesso sistema voltato (Carbonara, 2000; Giuffrè, 1990; Carbonara, 1990).

Le modalità di trattamento dei sistemi di finitura superficiale d'intradosso restano purtroppo affidate a 'scelte di buon gusto' ovvero a 'mode', orientate sovente verso:

1. la rimozione con sabbiatura degli antichi scialbi di calce, con la messa a nudo della pietra a vista (scelta impropria da evitare);
2. il 'ripristino' di una pseudo velatura protettiva, che spesso avviene con l'uso di 'pitture acriliche' poco compatibili con i caratteri chimico-fisici del substrato murario.

Questo tema merita una seria riflessione, al fine di valutare la messa a punto di criteri e di metodi di restauro critico-conservativo dei sistemi di finitura superficiale (a latte di calce? ad olio? a tempera?) che suggeriscono futuri scenari d'intervento.

Al fine di formulare metodi e strumenti da impiegare per restaurare staticamente le volte stellari salentine l'indagine approfondisce lo studio delle tipologie d'intervento compiute tradizionalmente sulla preesistenza volta-

ta e muraria locale[12]. Le proposte maturate sono ancora una volta il risultato di quanto osservato e sperimentato in cantiere, relazionando costantemente dati di natura tecnico-pratica e teorico-storico-documentaria[13].
Il restauro dei sistemi voltati salentini è compiuto nel passato con tecniche d'intervento dettate dalla logica pratico-funzionale del maestro muratore, semplici da eseguire e puntuali. In relazione alle cause che generano il degrado della materia e il deperimento delle strutture, l'intervento interessa la superficie voltata o la sua struttura portante. La sintassi linguistica adottata dai maestri lapicidi del XIX e della prima metà del XX secolo non differisce di molto da quella usata dai loro padri. Si registra, al contrario, uno stretto legame e una continuità di linguaggio tecnico e tecnologico fra gli interventi del XVI e quelli d'inizio XX secolo. Spesso, nella lettura dei contratti notarili di fine Ottocento o di inizio Novecento viene esplicitata l'esigenza di dover "restaurare" i sistemi "lamiati", perché cadenti e in stato di rudere. La causa principale del precario stato di salute dei sistemi portanti e portati è attribuita all'azione degradante dell'acqua piovana e dispersa sugli estradossi murari. Queste tipologie di danno invocano le seguenti e principali categorie d'intervento, realizzate nel passato sulle preesistenze dai maestri di volta e riassunte nel seguito come interventi chirurgici e di superficie:

1. scialbatura periodica delle superfici d'intradosso e estradosso, alle volte anche del battuto di terra superiore;
2. stilatura dei giunti e verifica periodica del loro stato di conservazione con materiali compatibili e propri della tradizione costruttiva locale;
3. bitumazione dei giunti dei lastricati delle terrazze praticabili (Arditi, 1888)[14].

[12] Nel 1997 la Scuola edile di Lecce ha organizzato il primo corso per formare operai specializzati nel restauro di edifici di particolare pregio artistico e architettonico del Salento, mentre nel 2000 si è tenuto il primo corso privato, organizzato a Campi Salentina (Lecce), per insegnare a costruire le volte *a spigolo*.

[13] Molte informazioni sono ricavate da fonti indirette d'archivio; spesso si tratta di contratti d'appalto fra committente e maestro muratore.

[14] Questa tecnica era molto diffusa nell'antico cantiere armeno e siriano, in cui le volte e le cupole erano protette con colate di bitume fuso, che isolava la pietra dall'acqua; nel secolo XII anche le colonne della chiesa dei SS. Nicolò e Cataldo in Lecce, in pietra leccese,

4. lavaggio delle superfici interne intonacate con acqua e cenere di carbone, soprattutto in prossimità degli spigoli, dove si annidano le spore e le macchie di condensa, praticato nei mesi primaverili;
5. 'allattatura' successiva alle operazioni di lavaggio, mediante intonaco a base di sola calce, acqua e polvere di marmo e tufina;
6. reintegrazione del battuto di terra dell'estradosso della volta dell'ultimo livello, con tufina a granulometria variabile, a seconda dello strato da costipare, calce aerea, cocciopesto, conchiglie, vetri e acqua;
7. revisione periodica delle canaline lapidee per lo scolo dell'acqua piovana, dei figulini dei pluviali in terracotta e delle bocche di deflusso delle acque meteoriche;
8. sostituzione delle *chianche* o dei coppi rotti e fessurati delle coperture.

In un territorio definito a basso rischio sismico gli interventi strutturali sui sistemi voltati mirano ad annullare le sollecitazioni prodotti dalle spinte fuori dal piano dei carichi permanenti e accidentali generati dal peso proprio delle stesse strutture. Le principali operazioni di consolidamento chirurgico sono:

1. zeppatura delle lesioni, mediante l'inserimento di cunei di pietra o di legno, opportunamente sagomati, in funzione di dimensione e andamento della lesione strutturale;
2. reintegrazione profonda dei giunti fra i conci eccessivamente erosi e polverizzati, con impiego di malta di carparo o di tufina e calce (la calce idraulica viene usata non prima del XIX secolo); la pozzolana non è mai impiegata in questa zona della Puglia; sono invece presenti cocci rotti e *uelu* (terra rosso armena);
3. sostituzione di *chianche* corrose o microfessurate, bioturbate o fratturate;
4. sostituzione totale del manto d'inchiancato se lesionato integralmente;
5. sostituzione dei canali per lo scolo delle acque;

furono imbevute di encausto, per evitare che gli agenti atmosferici le degradassero.

6. sostituzione parziale dei conci dell'*appesa* (azione di cuci e scuci);
7. sostituzione chirurgica di conci della vela o delle punte della struttura voltata a spigolo o a squadro (cuci e scuci), con successive operazioni di finitura superficiale, compiute con la tecnica della 'confrontatura';
8. incatenamento delle strutture voltate mediante l'inserimento di tiranti metallici inchiavardati con capochiavi alle murature portanti di perimetro (Arditi, 1888; Carbonara, 1992)[15].
9. rifacimento integrale della volta. Questo caso interessa raramente le strutture voltate salentine, se soggette ad una corretta e continua manutenzione ordinaria.

Si osserva, pertanto, che le volte soggette a continua manutenzione resistono molto bene alla prova del tempo, anche perché non sopportano grandi carichi e sono di ridotte dimensioni d'impianto (3x3; 4x4; 5x5 metri)[16].

4. *Conclusioni*

In conclusione, gli interventi compiuti nel passato sulle preesistenze sono dettati in primo luogo dalla cultura di una manutenzione costante e continua di tutti gli elementi dell'edificio. Questo approccio è supportato fino al 1955 ca dalla profonda conoscenza che le maestranze hanno delle pratiche del cantiere tradizionale.
Oggi si rileva frequentemente l'indotto abbattimento dei sistemi voltati stellari (direttamente, a colpi di piccone; indirettamente, mediante una volontaria assenza di manutenzione atta ad indurre crolli intrinseci), motivando la sostituzione integrale con una presunta inefficienza statica delle antiche strutture voltate.
La demolizione delle volte leccesi è causata nella maggior parte dei casi

[15] Il sistema dell'incatenamento, già sperimentato da Rondelet alla fine del Settecento, viene dimensionato per le volte leccesi solo alla fine del XIX secolo.

[16] La serie d'interventi sopra descritti non sono mai specificati nei documenti d'archivio, in cui l'omissione descrittiva sembra volontaria, dettata da una pratica d'intervento consolidata e diffusa a tal punto da non richiedere ulteriori specificazioni. L'unico termine ricorrente è quello del "lamiare", "rilamiare", "girare a volta", "sostituire" i conci ammalorati.

da una scarsa conoscenza della capacità resistiva dei sistemi stellari, da una diffusa ignoranza in merito ai materiali della tradizione edilizia locale; dall'esigenza di 'bruciare' i tempi del cantiere edile in poche settimane; dalla carenza di manodopera specializzata; dalla difficile applicazione del Capitolo 8 della Circolare Ministeriale del 2008, riferita ad interventi strutturali locali 'di riparazione' in edifici storici, per i quali risulta difficile tradurre l'efficienza strutturale di sistemi murari e voltati in modelli strutturali verificati.
Quando non abbattuti, le azioni compiute oggi sui sistemi voltati risultano pertanto contaminate dall'uso di materiali e tecniche attuali di cantiere (cordoli in cemento armato e malte cementizie di allettamento o di rigenerazione) poco compatibili con la preesistenza; oppure dall'introduzione nelle fasi lavorative di sequenze procedurali importate da squadre di giovani maestranze edili, provenienti soprattutto dall'area balcanica o nord africana.
Oggi non è ipotizzabile un ritorno alle origini della pratica di cantiere antico, perché altro è il periodo storico e altre sono le esigenze, non trascurabili, di varia natura (materiale, tecnica, tecnologica, economica, normativa e sociale). Appare anacronistico pensare alla sopravvivenza di un artigianato locale, che sia espressione di valori culturali (sociali, tecnici, religiosi) non più attuali. Pertanto, bisognerà ripensare la formazione di una nuova forma d'artigianato, insegnando i segreti delle tecniche costruttive tradizionali e un impiego consapevole di conoscenze antiche e di nuove tecnologie.
Nella storia dell'architettura di Terra d'Otranto il fenomeno delle volte stellari s'inserisce all'interno di un'interessante sperimentazione estetica, semantica, economica e strutturale. È figlio di una civiltà contaminata da influssi e sinergie diverse, provenienti da Oriente e da Occidente, suggerite non solo dalla storia ma anche dai caratteri litologici dalla materia prima da costruzione (pietra calcarenitica) e dall'estro dei maestri 'di volta' e 'di muro'. I nuovi sistemi voltati rivoluzionano la spazialità interna delle architetture tanto quanto la sintassi linguistico-decorativa dei *tableaux* 'barocchi' e delle soluzioni d'angolo contraddistingue i fronti stradali delle città cinque-settecentesche **[fig. 6]**.
Questa ricerca, condotta su piani differenti (di natura storico-architettonica, strutturale, chimico-fisica, tipologico-formale, tecnico-costruttiva, di dettaglio decorativo ecc), individua nella sinergica compartecipazione di diversi fattori l'avvento di questo nuovo sistema di orizzontamento, declinato nelle sue infinite variazioni del tema stellare.

A distanza di anni, l'indagine lascia ancora irrisolte molteplici questioni, quali ad esempio la mancata dimostrazione di rapporti diretti e documentati fra l'attività delle maestranze valenziane e quelle salentine; se, come, ad opera di chi il sistema voltato stellare sia stato introdotto nel Salento; quando (alla fine del XV secolo o nella prima decade del XVI secolo?); il ruolo culturale svolto dagli ordini religiosi nei secoli XVI-XVII con l'avvio del processo di latinizzazione dei territori grecanico-bizantini; i legami e i nessi degli appalti avviati in Terra d'Otranto nei secoli XVI-XVIII; le modalità di diffusione del fenomeno costruttivo e la storia civile ed economica; il ruolo delle committenze regie rispetto quello della potente nobiltà locale; i motivi della mancata citazione di questa tecnica nei capitolati di appalto consultati negli archivi storici diocesani, comunali e provinciali[17].
La ricerca, pertanto, continua nei cantieri sperimentali di restauro critico-conservativo.

[17] Sono stati consultati gli archivi storici comunali di Ostuni, l'archivio storico diocesano di Brindisi-Ostuni; l'archivio storico diocesano di Oria; l'archivio di Stato di Brindisi e di Lecce.

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Fig. 1. Costoloni della campata quadrangolare, voltata a crociera, nel presbiterio turrito della Collegiata di Manduria (TA) dedicata alla Santissima Trinità. Dettaglio del concio di chiave e dell'innesto dei conci rispetto il fondo del sistema voltato (secolo XIII?; foto 2020, Ilaria Pecoraro).

Fig. 2. Collegiata di Manduria (TA) dedicata alla Santissima Trinità: dettaglio dell'abside cinquecentesco polilobato, in pietra calcarenitica lavorata con conci a sezione variabile, con statue decorate con foglia d'oro e costoloni disposti a raggiera con sezione variabile, decorati da roselline indorate (foto 2021, Ilaria Pecoraro).

Fig. 3. Collegiata di Manduria (TA) dedicata alla Santissima Trinità: Cappella di Costantinopoli (struttura voltata a stella della seconda metà del XVI secolo; decorazione a stucco post sisma del 1743; foto 2021, Ilaria Pecoraro).

Fig. 4. Convento degli Scolopi in Manduria (TA): sistema voltato a stella su impianto rettangolare irregolare, con finitura superficiale a scialbo di calce e sesto acuto (1696-1745; foto 2021, Ilaria Pecoraro).

Fig. 5. Chiesa Madre di Carovigno (BR), dedicata a Santa Maria Assunta in cielo (XIV-XIX secolo): particolare dell'intradosso del settecentesco sistema voltato stellare composito ad ombrello (foto 2021, Ilaria Pecoraro).

Fig. 6. Chiesa dei Santissimi apostoli Pietro e Paolo, detta della Madonna del Carmine in Manduria (TA) attribuita all'arch. Mauro Manieri: sistema voltato stellare composito, ricavato nel volume posto ad intersezione fra la scatola muraria perimetrale e l'aula liturgica. Quest'ultima, ad impianto ovale è coperta con sistema voltato stellare sorretto da squadri simmetricamente distribuiti, con finitura superficiale a stucco e sesti acuti (1688-1741; foto 2021 Ilaria Pecoraro).

Tullia Iori

Reinforced concrete in Italy: a sweeping story*

In the following years, while starting to work at the University of Rome Tor Vergata where I am now full professor, I continued to deal with the history of construction and the history of engineering in Italy. In particular, from the results of the PhD, I deepened the figure of Pier Luigi Nervi, publishing some books (Iori, 2009; Iori, Poretti, 2008) and co-curating an exhibition on his work at the MAXXI Museum in Rome: *Pier Luigi Nervi. Architettura come Sfida. Roma. Ingegno e costruzione* (Iori, Poretti, 2010).

At the same time I started to deepen the history of Italian engineering in the post-war period, with particular reference to the magic years of the economic boom. The Italian structural engineering in those years was the protagonist at world level, thanks to major public works such as the Autostrada del Sole, and major events such as the Rome 60 Olympics and the celebrations for the 100th anniversary of the Unification of Italy in 1961 in Turin.

In 2011, with Sergio Poretti, we started the research project SIXXI - XX Century Structural Engineering: the Italian contribution, funded by European Research Council Advanced Grant 2011 (ERC Adv Grant, www.sixxi.eu).

The general goal of the new research is to give a major contribution to the international history of the role of engineering in architecture.

* The book that won the second edition of the Edoardo Benvenuto Prize was the result of doctoral research (1997-1999; PhD programme "Architecture and Construction", University of Rome "Tor Vergata"), conducted under the guidance of Sergio Poretti (1944-2017), my maestro. The research was published in 2001 in the Edilstampa "Il modo di costruire" Series (Iori, 2001).
I did not have the opportunity to know Edoardo Benvenuto personally but I was familiar with his writings. I considered the book The science of construction and its historical development as a sacred text. When my Phd thesis won the prize, it was a strong emotion and I really thought I was on the right track.

My sincerest gratitude goes to the Associazione Edoardo Benvenuto for the invaluable promotion of research in science and construction art and to the jury of that edition: Luigi Gambarotta (Università di Genova), Karl-Eugen Kurrer (Berlin), Patricia Radelet (Université de Louvain-La Neuve).

The research project focuses on the works and protagonists of twentieth century structural engineering in Italy. Started in march 2012, the SIXXI project involved the team until the end of February 2017.
After Sergio's death, I sadly continued on my own to develop the research, and continued to disseminate the results at all levels with dozens of SIXXI lectures. The results achieved so far are collected in the first 5 volumes of the SIXXI series, which includes a photostory with images of Italian engineering between the nineteenth and twentieth centuries (Iori, Poretti, 2014; Iori, Poretti, 2015a; Iori, Poretti, 2015b; Iori, Poretti, 2017; Iori, Poretti, 2020).

The following is a brief summary of the awarded book, which dealt with the history of reinforced concrete in Italy in the twentieth century. How did this new technique begin to spread at the end of the last century? To what extent was this new technique imported from abroad? And how did foreign systems set off its autochthonous development? What role did the post-earthquake reconstruction of Messina and Reggio play in this? Moreover, what role did Italian researchers occupy in the international debate on the adaptation of a classical theory to a non-homogeneous material? And subsequently, how were the limitations placed on the use of steel during Italy's autarchic period overcome? How did the development of reinforced concrete continue? And while this technique became widespread in daily construction work, how did it develop in the experimentation of greater works?
Based on previously barely utilized sources from the Patents Office Archives and specialized engineering magazines, this study reconstructs the use of reinforced concrete in construction work in Italy from its first uses up to the Second World War. The study focuses on a sequence of more than one thousand inventions, both Italian and foreign patents, which determined the many formulations and subsequent revisions of this technique. It attempts to bring order to a vast debate on the structural theories and calculation methods, also employed in Italy, which accompanied the development of reinforced concrete.

The advent of a new technique (1850-1900)

Between 1850 and 1900, experiments were conducted, both in Europe and in the United States, on the use of a new material created by combining two very widely employed industrial products: cement and structural iron. This new material was called *béton armé*.

How was Italy involved in this experimentation?
Construction techniques in Italy were certainly unique. Although the general lag in industrialization, and consequently in the iron and steel industry, had not completely blocked the development of iron architecture, it certainly slowed it down. Construction work, however, was undergoing a period of interesting evolution after many centuries of relative standstill. In Piedmont, in particular, Alessandro Antonelli rationalized reinforced masonry in boldly static monuments. Cement, which was just beginning to be produced industrially at this time, was exclusively used for decorations and finishings through a technique known as “artistic cement”.
In fact, Italy did not directly participate in the pioneering experimentation phase of what was referred to as *béton armato*. Nonetheless, the construction industry was very interested in this new technique. Thus, the most important patents protecting this technological improvement were also deposited in Italy, although the divulgation and application of these new techniques was much slower.
François Hennebique, a shrewd entrepreneur holding an innovative patent, played a key role in the diffusion of this new material, especially in Italy, where he set up a very efficient series of branch agencies.
In the meantime, the increasingly daring reinforced concrete applications in Europe set off an animated debate, primarily involving French and German scientists, who focused on the formulation of a less empirical “rational” calculation theory for the new material than those that had been previously made by the patent owners.
The importation of foreign systems led to widespread experimentation, which soon engaged the entire local enterprise system. By the mid 1890s, a vast number of local patents had been deposited that often were sophisticated variations of the already tested foreign systems.
The application of these new techniques was not limited to the leading construction firms that had quickly converted to the new technology. Newly created companies specialized in the use of reinforced concrete construction were also quick to make the most of the new development. The number of buildings and monuments created, both with Italian and foreign patents, right before the turn of the century demonstrates the interest of local enterprises in this new material and its incessant diffusion.

Diffusion (1900-1915)

By the turn of the century, the pioneering period of reinforced concrete had come to an end both in Italy and in Europe. It was officially recog-

nized as an invaluable material and was commonly used in construction work.
The new construction technique became part of the professional training of engineers. Manuals that addressed the main issues related to the use of reinforced concrete and established basic procedures and standards for its application were published. In universities, science and mechanical construction courses embraced the technique and graduates were finally equipped with the information necessary to safely apply the various systems.
At the same time, mainly due to the still many computational uncertainties, the increase of uncontrollable construction systems and the collapse of buildings under construction, many European countries adopted a series of cautionary measures, which raised further debate. In Italy, the Ministry of Public Works approved a series of regulations governing reinforced concrete construction work to guarantee the safety of public works.
However, the diffusion of the new material and its widespread use in the construction of residential buildings and public infrastructures required a new national law. The thousands of structures erected by Hennebique agencies were mirrored, in the many new technical magazines dedicated exclusively to reinforced concrete construction work, by the applications carried out by the many specialized construction companies in constant competition with one another.
In the meantime, once the framework-concrete mechanism had been firmly established, patent experimentation turned to individual construction elements. In particular, floor construction techniques, in which reinforced concrete was used to replace the traditional construction methods. The new inventions showcased the first prefabricated beams and the first hollow clay blocks for the construction of mixed element floors, which were soon to undergo an extraordinary development in Italy.
A decisive stimulus for the definitive affirmation of the reinforced concrete technique came in the aftermath of the 1908 earthquake in Messina and Reggio Calabria. The anti-seismic framework became widely employed in the twenties and profoundly conditioned the way in which reinforced concrete was employed in Italy.
In the meantime, the classical computation theories, which had since been perfected, were employed in the erection of large works. The *Risorgimento* Bridge, erected in Rome in 1911, was to influence many of the theoretical debates of the coming years. It played a key role in comprehending the limits of the elastic behaviour hypothesis of reinforced concrete. The con-

struction difficulties of the bridge forced Hennebique himself to overcome the limits imposed by the patent and to use reinforced concrete freely, fully exploiting its infinite possibilities.

Standard use of reinforced concrete (1915-1935)

Starting with the post-war reconstruction and throughout the nineteen-twenties, the use of reinforced concrete construction had become a common construction technique in Italy.
In this phase, characterized by many public residential buildings, this construction system, which proved faster and cheaper than traditional systems, turned out to be perfect and was often used together with load-bearing masonry in a mixed solution.
The reinforced concrete technique was no longer the exclusive domain of patents and specialized firms. It had become part of mainstream professional engineering and was employed by many small and medium sized companies. In order to simplify the work of neophytes and expedite expert projects, (not always rigorous) manuals, abacuses and charts became readily available along with new mechanical tools such as slide rules and the first calculators.
The fact that the this technique became increasingly accessible to companies and project designers, who were not necessarily qualified, called for a more scrupulous appliance of the existing laws and, in particular, the regulations regarding executive project aspects. The outdated law issued in 1907 was reviewed in 1927 and extended to apply to all construction work, both public and private.
In the meantime, the scientific community continued to study the material and refine the knowledge of previously ignored parameters. Industry continued to improve and transform production techniques, privileging artificial cement over natural cement, which had grown scarce. This process of refinement lead to the invention of special use cements.
During the latter half of the nineteen-twenties, a series of circumstances drastically changed the course of Italian construction techniques. And the role of reinforced concrete changed, too. Within a couple of years, in fact, the Fascist regime had drawn the construction sector back into a state of corporatism. The reaction to the great crisis of 1929, which had interrupted the development of construction work, spawned the development of new techniques. Finally, this all took place concomitantly with the beginning of the great debate on modern architecture.

What role did the reinforced concrete technique play in this transformation process? And how did the reinforced concrete technique evolve as a result of this transformation?
While technological experimentation was directed towards large structural works, research investigated the shape of reinforced concrete frameworks in relation to the search for new forms and worked towards the definition of a new architectural language. Construction methods evolved rapidly and involved architectural culture. The results of this phase are witnessed in the works carried out in the first part of the nineteen-twenties, before the autarchic period changed the conditions once again.

Autarchic experimentation (1935-1943)

The Italian aggression of Ethiopia and the subsequent economic sanctions raised against Italy by the Society of Nations, in November 1935, gave rise to a critical phase in the economic policy of the Fascist regime. The protectionist orientation aimed at making Italy self-sufficient, which was part of corporatism, became increasingly stringent and lead to the most intense phase of autarchy, which was also motivated by the subsequent military decisions.
In construction work, as in all other productive sectors, the objective of becoming independent from all imported material became a determinant factor. This sparked a fierce debate, but also increased the range of experimentation aimed at finding new solutions with a greater "national value." The use of reinforced concrete was dependent on foreign supplies both for the wood, used for the moulds, and for the framework iron. Thus, the technique was accused being anti-autarchic. Notwithstanding the fact that the "gold-cost" ratio revealed that it was more convenient than other building techniques (using steel or load-bearing masonry), the need to reserve all iron for the military effort condemned it. The use of reinforced concrete became severely limited from 1937 and by 1939 was completely banned.
However, the restrictive measures did not stop engineers and researchers, who continued to study ways to make reinforced concrete more "autarchic". There were two main lines of research.
A more "traditional" line of research turned to a long-term approach and hypothesised that once the war was over, the use of reinforced concrete would pick up again with nationally produced iron. Thus, research was aimed at economizing its use in frameworks by employing higher precision

calculation methods, greater project care and the use of materials with higher performance potential (high resistance cement and steel).
The other line of research, which was more innovative and aimed at reaching immediate results, returned to the experimentation that had been carried out right after the war, when supply problems and the high cost of traditional building materials had fuelled research on alternative, nationally-available materials. As had already been proposed during the period of "building frenzy", wood and new materials, such as bamboo, asbestos-cement and aluminium, were proposed as possible materials for building the frameworks. In order to reduce dead weights in buildings, the use of cement blocks and perforated bricks was increased together with pumice-based elements and materials composed of other waste products. A law was also passed to allow the construction of mixed-element floors without slabs and horizontal construction methods with reduced frameworks were studied. As the lack of framework iron increased, experimentation turned to the construction of floors without iron. From the Miozzi to the Neumann patent, dozens of systems based on the traction resistance of bricks and cement claimed floor construction stability without the use of iron. This final phase of research, which was abandoned right after the war, often provided unexpected results that were not justified either by calculation or material characteristics.

Great works (1920-1943)

Starting in the mid nineteen-twenties, as we have seen, reinforced concrete gradually became an everyday construction material. The innovative turmoil that had accompanied the development of this material shifted from construction elements to application methods as part of the renewed building conception that progressively lead to a complete transformation of the existing relation between load-bearing masonry and framework.
So, what was happening in the great works sector? In order to answer this question, we have to step back to the beginning of the nineteen-twenties and follow the experimental course of the building techniques relevant to structurally important works.
Manuals and experience were sufficient to erect normal reinforced concrete buildings; technological and theoretical experimentation was directed at larger structures. In fact, this was what interested the specialized firms, which left ordinary construction work to the many small companies that had blossomed, in order to concentrate on projects in which accumu-

lated experience was indispensable. The most recent industrial and theoretical inventions, from high-performance artificial cement to building site machines and from sophisticated frameworks to intricate calculation theories were applied to these projects.
Starting in the mid nineteen-thirties, the Fascist regime's economic policy condemned the use of the little available autarchic reinforced concrete and eventually forbid its use in both public and private civil constructions. However, if this ban had been rigorously applied to the great works sector the building sites would have ground to a complete halt. In these cases, the projects were reviewed in order to minimize the use of iron. On the one hand, the resistance values of materials were arbitrarily increased and isostatic structures were employed in order to allow preciser calculation and, consequently, optimal member use. On the other hand, a process of involution set in: bearing distances diminished and the use of reinforced concrete shifted to “less reinforced” concrete and then to simple concrete. Finally, *forte sesto* arches were made in traditional masonry as it was considered more autarchic.
Although autarchy brought about an involution of practical applications, it also stimulated research into rational structures, which according to the fundamental principles of structural engineering also means less expensive structures. Old solutions that had been put forward in the pioneering phase or this material resurfaced against the attempts to substitute or eliminate iron as new technological developments (new cements, special steels) and the increased knowledge of these materials (cement traction, framework release and viscous phenomenon) allowed these techniques to be fully developed.
The objective of autarchic experimentation was to radically transforming the relation between iron and cement and overcome the intrinsic limits of reinforced concrete. This research followed two routes.
One route was represented by the thin structures that starting with Lambot and Monier's cemented iron and European research on vaults and shells brought Pier Luigi Nervi to experiment on highly-reinforced reduced thickness structures. This lead Nervi to the invention of *ferrocemento* a new homogeneous, isotropic and elastic material that would characterize his entire post-war production.
The other route also led to the creation of a new material, which was erroneously called “pre-compressed reinforced concrete” in Italy. Research conducted at the beginning of the century was used to invert the role of the two basic materials of reinforced concrete: iron no longer had to ab-

sorb the traction that the cement couldn't support, but became the means to impress the conglomerate with the necessary constraint to absorb all the stress.
A lively debate was raised in Italy regarding the far more efficient experimentation conducted in Europe by Eugène Freyssinet, Franz Dischinger and Ulrich Finsterwalder, which lead to important theoretical contributions such as those made by Gustavo Colonnetti. In the years preceding the war, he set the premises for important developments that were to be widely used during the reconstruction period and which were to decree international recognition for another Italian engineer, Riccardo Morandi.

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Fig. 1. Cover of the book awarded with the 2001 Edoardo Benvenuto Prize.

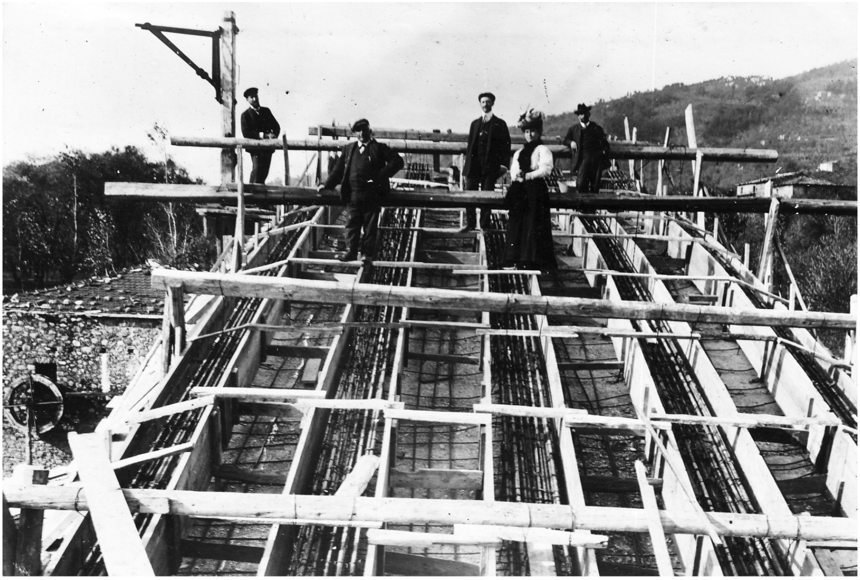

Fig. 2. A. Muggia, Bridge over the Magra River at Caprigliola and Albiano, 1903-1908 (Private Archive Nino Ferrari).

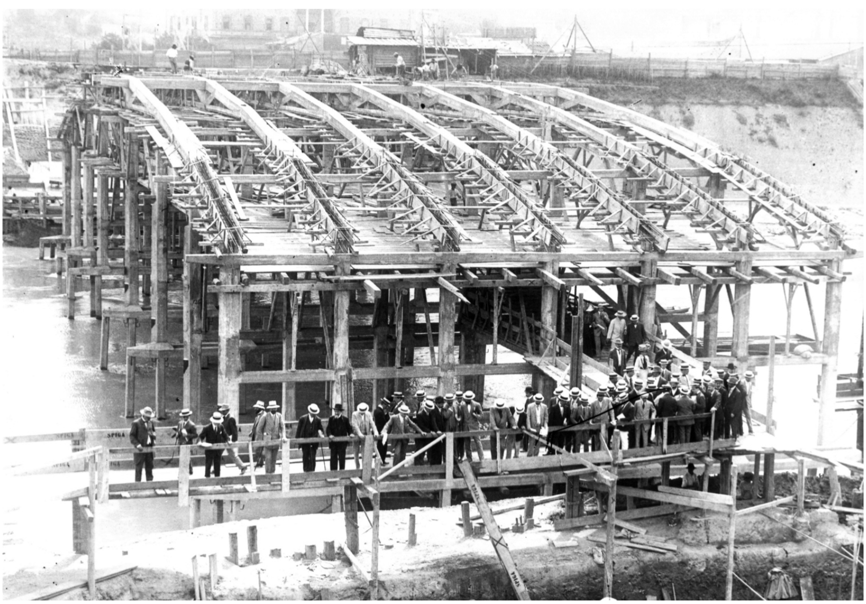

Fig. 3. Risorgimento bridge during construction, 1911 (G.A. Porcheddu Archive, Turin Polytechnic).

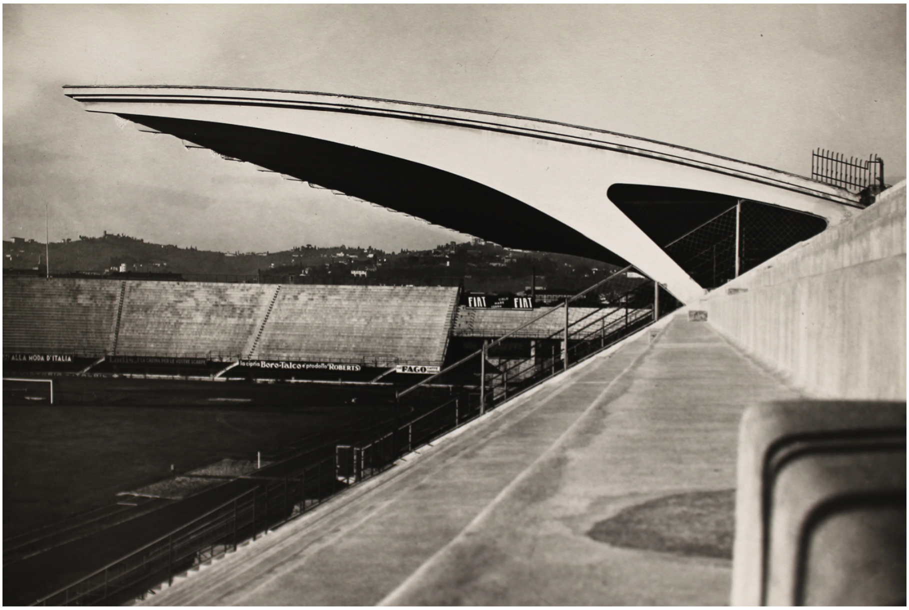

Fig. 4. P.L. Nervi, Stadium in Florence, 1931 (Photo Archives Touring).

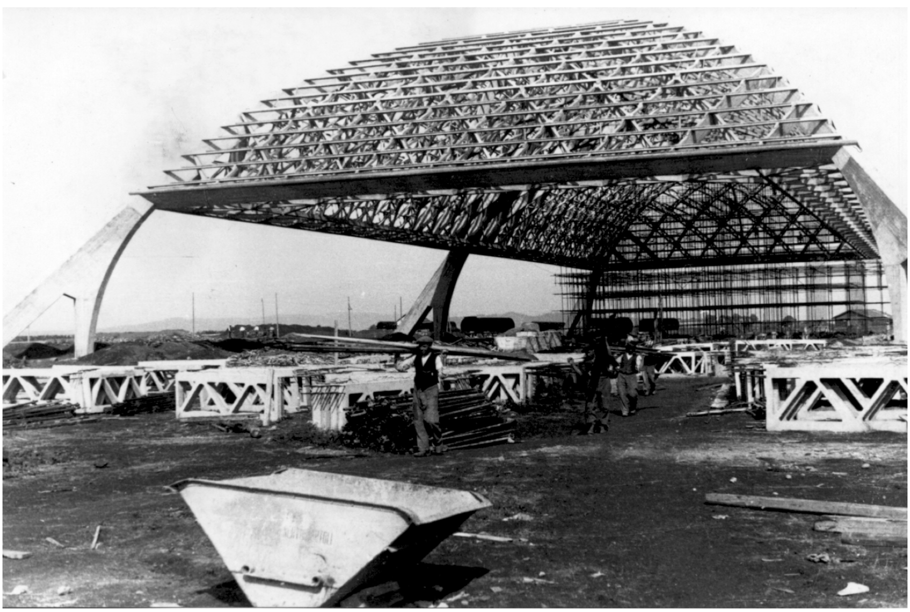

Fig. 5. P.L. Nervi, “Second series” Hangars, Orbetello, 1939-1942 (Maxxi Architecture Archives, P.L. Nervi collection).

Fig. 6. P.L. Nervi, Magliana Pavillion, Roma, 1944-1945 (Photo Sergio Poretti).

John Ochsendorf

The stability of a masonry arch on buttresses*

This research investigates the stability of a masonry arch supported on buttresses and the conditions necessary for collapse to occur. The safety of the system is examined by studying the influence of imposed displacements. As the buttresses lean, the thrust of the vault increases and the resistance of the buttress decreases. The collapse mechanisms are identified for the static case of leaning buttresses and this analysis illustrates that the arch will collapse and the buttresses will remain standing in most cases.
Medero *et al.* (1998) used finite-element methods to study the nave of the church in Vézelay, which Viollet-le-Duc had studied extensively. By allowing for the masonry to separate in the absence of compressive forces, the authors predicted a likely collapse mechanism and the mode of deformation in the arch. In addition, they were able to model the hinges in the vault as well as some cracking in the buttress. The analysis indicated that a fracture may form in each buttress as the structure nears the collapse state. The study used sophisticated gap elements to model the response of the mortar in the joints, but it has a number of shortcomings. In particular, the authors analysed the undeformed shape of the vaults, in an effort to explain the deformations, and did not analyse the deformed structure as it exists today. Conventional finite-element formulations have difficulty with the large-displacement behaviour and geometrical non-linearities of this type of problem. Furthermore, the results are extremely sensitive to the assumptions about the material proprties and boundary conditions. Finally, the authors did not explore the implications of imposed displacements for the stability of the structure, though they conclude that the stability is "very sensitive to abutment movement" (Medero *et al.*, 1998).
Viollet-le-Duc (1854) had studied the same church in the 19th century and

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wrote about the large deformations of the vault **[fig. 1]**. In this structure the vaults of the lateral naves help to support the main buttress, but the gross deformation in the central nave and the leaning of the buttresses is apparent. Medero *et al.* (1998) sought to explain the deformations with complex material models using finite-element analysis. In contrast, this dissertation assumes that the material does not deform and that the structure deforms according to rigid-body mechanics by formation of hinges between rigid elements and by support movements. The goal is to find the maximum load or displacements that would cause collapse.

Even with the simplified material assumptions of limit analysis, the structural behaviour of buttressed arches is complex. Increased deformations can lead to greatly increased internal forces. A first-order structural analysis, based on theories of small deformations, will not reveal the sensitivity to additional movements that determine the safety of the structure. Huerta and Lopez (1997) considered the implications of additional movements for the stability of the structure, which is straightforward to consider because the system is statically determinate. The arch or vault has three hinges, and the thrust in the actual vault can be determined uniquely. The resistance of the abutments to this thrust can be estimated, and a clear understanding of the structural safety emerges. Finally, it is possible to impose increased displacements by artificially leaning the buttresses further, and to investigate the conditions that will lead to collapse of the structure. Using principles of limit analysis, it is straightforward to determine the collapse state of a rigid-block structure due to an applied load. The hinge locations are postulated, and a work calculation is performed to determine the maximum applied load, P. The critical collapse mechanism will occur for the lowest value of applied load which forms a kinematically admissible mechanism (Heyman, 1969; Livesley, 1978). For an arch on masonry buttresses, the analyst must consider collapse mechanisms involving a combined arch-buttress mechanism, including the possibility of a fracture surface in the buttress, as in figure 2.

If the buttress were so massive as to provide effectively rigid abutments to the arch, collapse of the arch could be considered in isolation. But in other cases, a combined "arch-buttress" mode may lead to lower values of the collapse load than for the arch alone. For masonry buildings, a concentrated load on top of the vault is unlikely, though Heyman (1993) has demonstrated that this can occur in the event of a heavy roof collapsing onto the arch or vault below. This dissertation does not investigate the influence of

applied loads, since previous researchers have investigated this topic in detail. For buttressed arches, it is sufficient to state that engineers must consider mechanisms involving the failure of the buttress by the formation of a fracture surface, as illustrated in figure 2.
Applied displacements are a significant threat to the safety of buttressed arches. Various kinds of displacement may de-stabilise the structure, and this paper will focus on the most common pathology: outward leaning of the buttresses. As the buttresses lean, the change in geometry alters the equilibrium conditions. The thrust capacity of the buttress decreases and the thrust of the arch increases due to the displacements. The current paper considers the collapse state of the buttressed arch system by combining the results from the dissertation by Ochsendorf (2002). The goal is to demonstrate the conditions for the failure of the system as a result of leaning buttresses and determine in a given case which component is more likely to fail first: the buttress or the arch.

Geometry of buttressed arches

Traditional masonry structures are built in a wide variety of forms, with different buttressing systems and endless variations in vaulting geometry. This dissertation adopts the simplest possible form: circular arches supported on rectangular buttresses. This system is typical of barrel vaults supported on rectangular masonry walls, or of individual masonry arches supported on individual buttresses. The chosen geometry of circular arches and rectangular buttresses is applied only to illustrate the general patterns of behaviour and the likely causes of collapse. In practice, each individual structure is unique and the analyst should apply the general methods to the specific structure being considered.
The paper investigates two particular extreme configurations for buttressed arches **[fig. 3]**. In each case, the width, *b*, of the buttress at its base is regarded as the unit of length.

Case A is a highly conservative configuration, with the buttress width equal to half the span, and the height of the arch support equal to the span. Furthermore the buttresses are very conservative with a height to width ratio of three. Case B is more daring, and is approximately the same as the proportions of the high transverse arches and slender buttresses of the Sainte-Chapelle in Paris (Heyman, 1996). The span of the arch is three times the width of each buttress, and the height of the arch is 1.5 times the span of the arch. In addition, the buttresses of Case B are much slenderer than Case A, with a height to width ratio of six. For both cases, the half angle of embrace, α, and the thickness ratio, *t/R*, are variable. Thus, the buttress systems of Case A and Case B can be investigated for a wide variety of arch geometries.

Arch on leaning buttresses

Many buttressed arches and vaults have collapsed throughout history, for example the partial collapse of the dome of the Hagia Sophia and the partial collapse of vaulting bays in many Gothic cathedrals. In most cases, the arch or vault collapses and the buttresses remain standing. The current section analyses the collapse on account of the lean of buttresses, and seeks to explain the collapse of vaulting and the apparent resilience of buttresses.

As the buttresses lean, the span of the arch increases. The capacity of buttresses for horizontal thrust decreases linearly with the rotation ϕ as the buttress leans. The arch will deform to accommodate the span increase, and the change in geometry will lead to increased values of horizontal thrust. In the case of buttressed arches, the progressive increase in the rotation of the buttresses will lead to increasingly unfavourable conditions until the arch collapses. To assess the safety of a buttressed arch, the engineer must determine if the arch will collapse first, or if the capacity of the buttress will be exceeded by the increased thrust of the arch. This section proposes a method for analysing the problem.

For arches with small thickness/radius ratios, collapse will occur for small values of span increase. For larger thickness ratios, the arch can resist greater span increases and will provide greater increases in horizontal thrust before collapsing. As the thickness/radius ratios increase further, eventually the arch thrust is sufficient to cause the buttress to fail, and the failure of the buttress will lead to collapse of the arch. The program ArchLean, written in Matlab, analyses the problem for a prescribed geometry of the arch-buttress combination by increasing the lean of the buttress until the arch collapses (Ochsendorf, 2002). The program is illustrated here with reference to the buttress configurations of Cases A and B **[fig. 3]**. For an angle of embrace of 120° (α=60°), the collapse state is determined for varying values of the thickness ratio, *t/R*. The results are presented in figures 4 and 5 for thickness ratios ranging from 0.05 to 0.20.

The change in horizontal thrust is plotted in Figures 4 and 5, normalised by the maximum thrust capacity of the vertical buttress, H_u. The horizontal axis is the angle of inclination of the buttress in degrees. Sketches indicate the left-hand buttress configuration at ϕ =0° and ϕ =5°, respectively. The arch of thickness ratio 0.10 is illustrated at the point of collapse due to excessive span increase.

For a given geometry, the program ArchLean calculates the resistance of the vertical buttress to horizontal loads. The buttress lean, *f*, is slowly increased, thereby decreasing the resistance of the buttress and increasing the span of the arch. The new geometry increases the thrust of the arch. In all cases the arches have been analysed for 1° voussoirs, so that the intrados hinge can move freely as the span increased.

The program terminates when one of the two collapse scenarios occurs: either the "strong-buttress" or the "weak-buttress" collapse mode. Each failure mode is defined as:

1. Strong-buttress: Failure occurs when the arch reaches the maximum span increase for the given geometry, and the arch collapses, leaving the buttresses intact (illustrated as ● on figures 4 and 5). In this case, the buttresses are "strong" enough to resist the increased thrust of the arch, but the arch collapses due to the span increase. All five hinges of the symmetrical collapse mechanism occur in the arch. This failure mode occurs for smaller thickness ratios in general.
2. Weak-buttress: Failure occurs when the buttress capacity for horizontal thrust is exceeded, the buttress rotates additionally, and the arch collapses (illustrated as ■ on figures 4 and 5). In this case the buttresses are too "weak" to resist the increased thrust of the arch, and collapse occurs when the buttress gives way. At this point, the arch will collapse, relieving the horizontal thrust on the buttress, and the buttress will remain standing. The symmetrical collapse mechanism has three hinges in the arch and one hinge at the outer edge of each buttress.

These two failure modes will be used to describe the collapse of buttressed arch systems due to outward leaning of the buttresses.

For Case A, the capacity of the vertical buttress is more than five times the initial minimum thrust of the arch: see Figure 4. As the buttresses lean, the arch will collapse before the thrust capacity of the buttress is exceeded for most thickness values, i.e. in a "strong-buttress" failure mode. But at very high thickness ratios, such as t/R =0.20, the capacity of the buttress may be exceeded. In this case the buttress is unable to resist the thrust of the arch, and the buttress fails. When the arch thrust exceeds the buttress capacity, a five-hinge mechanism forms in theory, with one hinge at the base of each buttress and three hinges in the central arch. In practice, only four hinges are required for collapse, and any asymmetry will cause one buttress to give way first. The buttress will begin to rotate outwards freely, and the arch will collapse. As a result, the horizontal thrust acting on the buttress becomes zero, and the buttress remains standing. Thus, for high thickness ratios, failure will occur by a "weak-buttress" mode. The geometry of the buttressed arch of Case A is highly conservative (as evidenced by the much higher capacity of the buttress in comparison to the thrust of the arch), so the system is able to withstand considerable angles of tilt without collapse. The conservative design of the buttresses means that in most cases failure will occur in the arch before the capacity of the buttress is reached.

For Case B, the capacity of the buttress is approximately three times the initial minimum thrust of the arch: see figure 5. The springing of the arch is more than twice as high as the springing in Case A, and small angles of tilt will lead to larger increases in the span length for the arch. In addition, the buttresses are slenderer and the higher centre of gravity will lead to greater reductions in the capacity as the buttress leans (since the capacity of the buttress decreases as the vertical centroid shifts horizontally). This is reflected in the greater negative slope (by a factor of about 2) of the buttress resistance for Case B compared to Case A. The relative slenderness of Case B is apparent, and the arch will collapse at less than 2° of buttress leaning for most thickness ratios. For thickness ratios above 0.10, the buttress capacity will be exceeded and the arch will collapse when the leaning buttresses give way, i.e. "weak-buttress" failure will occur. Thus, in Case B, weak-buttress failure is more likely due to the more daring design of the buttressing system.

From these two examples, the behaviour of the structural system is clear. For most arch thickness ratios, "strong-buttress" failure will occur and the arch will collapse before the buttress. But for large thickness ratios, the thrust of the grossly-deformed arch may exceed the capacity of the leaning buttress, in which case the buttress will give way and the arch will collapse in a “weak-buttress” failure. In all cases, the buttress will remain standing after the collapse of the arch, due to the absence of the thrust from the arch. Thus, deformation of the buttress by leaning will lead to the collapse of the arch, but the removal of the thrust from the arch ensures that the buttress survives. For the buttress to overturn in static conditions, the centroid of the buttress must approach the horizontal coordinate of the hinge about which overturning will occur. This is not possible because the arch will collapse well before the buttress resistance approaches zero horizontal thrust. (This is the maximum amount of leaning a buttress can withstand, defined as ϕ_{max}.)

In order to investigate the influence of the angle of embrace of the arch on the behaviour of the system, additional values of α have been considered. Cases A and B have been analysed for angles of embrace 2α of 120° and 160° (α=60° and 80°, respectively), with the thickness ratio varying from the minimum thickness to a very large value of t/R=0.20. The results have been summarised in figure 6, which illustrates that greater angles of embrace will lead to collapse for smaller values of buttress leaning. In each of the two specific buttress systems A and B it is possible to determine the threshold thickness beyond which the buttress capacity will be exceeded. For Case A it is t/R=0.156, and for Case B it is t/R=0.104 for α=60°, as indicated on figure 6.

This threshold thickness is presented for a range of angles of embrace in **figure 7**, marking the limit between the two failure modes depending on the geometry of the arch.

An arch on spreading supports will actually collapse for slightly smaller span increases than those predicted by rigid body analysis. For the current problem this suggests that collapse will occur at slightly lower values of lean than predicted, because the grossly-deformed arch will exist in a precarious state. Any small movement would lead to the collapse of the arch. In actual structures any shift in the buttress due to the formation of the fracture or a sudden increase in leaning could lead to the collapse of the arch at lower values of lean than predicted.

In summary, for collapse due to leaning buttresses the arch may collapse before the buttress capacity is reached, in a mode defined here as strong-buttress failure. Alternatively, failure may occur by a “weak-buttress” mode, in which the horizontal thrust capacity of the buttress is exceeded. For relatively tall buttresses, such as those presented in Case B, collapse of the arch may occur for a small amount of leaning, even less than 1°, for typical arch geometries. In both cases, the arch will collapse and the buttress will remain standing, since the arch will no longer exert a horizontal overturning force.

Case study of the Capella da Nossa Senora do Monte

The 16th century Capella da Nossa Senora do Monte in Old Goa is one of the first monuments built by the Portuguese in the Indian subcontinent, and it has important historical significance. Deshpande and Savant (2001) recently carried out restoration work and added exposed steel ties to reduce the thrust of the vault on the masonry walls. The structure of this church provides a simple case study to illustrate the methods presented in this paper. This case study will examine the safety of the structure and seek to determine if the addition of steel ties across the vault was indeed necessary.

Structural description

The chapel consists of a barrel vault supported on rectangular masonry walls, and is constructed entirely of laterite stone derived from weathered basalt. The vault spans approximately 9.0 m and is supported on solid walls of approximately 2.7 m thickness (Figure 8). The barrel vault on the rectangular walls can be analysed for a 1-metre wide section along the nave, which is approximately equivalent to an arch supported on rectangular buttresses. To determine the magnitude of the forces in the structure, the unit weight of the material is assumed to be 25 kN/m^3.

In the current configuration, the South wall of the church (shown on the left in figure 8) is leaning by nearly 0.1 m at the top of the wall, which is equivalent to a lean of approximately 0.4° (Deshpande, Savant, 2001). The North wall is supported by the adjacent structure and remains nearly vertical. As a result of the leaning of the South wall, the span of the vault has increased and the vault has deformed by forming three hinges (A, B, and C in figure 8). For the purposes of the current paper, which aims to demonstrate the various measures of safety for a buttressed arch, the structure will be approximated as a circular arch supported on rectangular buttresses to simplify the analysis: see figure 8b. To simplify the problem further, the weight of the timber roof will be neglected for the current analysis. In addition, the effect of various openings in the walls, including several doorways and a number of circular openings, will not be considered. A thorough structural analysis would account for these factors, and would use a more accurate geometrical approximation of the arch and the buttress. The current study aims to determine the general equilibrium conditions, which can be quantified by using various measures of safety, and does not purport to be an exact analysis of the problem. Indeed, it will be demonstrated that an approximate analysis can be used to define the safety to sufficient accuracy.

The vault is circular with a radius of approximately 5.0 m and a thickness of approximately 0.5 m. The vault can be approximated as a circular arch subtending 120° with a thickness ratio of t/R =0.1, as in Figure 8b. For a 1-metre wide section of the vault at Goa in its original conformation, the minimum thrust is found to be 39 kN, corresponding to intrados hinge locations at 54° from the crown. The computed hinge locations of 54° compare well to the actual hinges in the vault, which are located approximately 50° from the crown. The vertical reaction at each support due to the weight of the vault is approximately 64 kN per metre width. For collapse due to spreading supports, the span must increase by approximately 8%. For the span of the vault at Goa of 9.0 m, this corresponds to a span increase of 0.72 m, which may be compared to the current increase of 0.1 m. At this collapse state, the thrust would have increased to approximately 2.2 times the initial minimum thrust of the arch, corresponding to a maximum possible horizontal thrust of 86 kN at the point when the vault will collapse due to spreading supports.

The buttresses are assumed to be rectangular, with a width of 2.7 m at the base and a height of 13.4 m, giving a h_b/b ratio of 5.0. The springing of the arch is assumed to be at a height of h =12.5 m, so that μ =0.9. The weight of the buttress is 905 kN per metre width, so considering the vertical reaction of the arch, V =64 kN, gives ψ =0.07 (where $\psi = V/W_b$). The capacity of the buttress for horizontal thrust is then determined from methods described in Ochsendorf (2002). The maximum horizontal thrust for the vertical buttress is approximately H_u =69 kN, corresponding to a fracture height of e =8.7 m. The reduction in thrust capacity as the buttress leans is approximately 10 kN per degree of leaning. Therefore, in the existing state with a lean of 0.4°, the buttress capacity has been reduced to H_ϕ =65 kN from 69 kN. To prevent failure of the buttress by sliding, the weight of the buttress above the springing is 61 kN and the weight of the arch provides a vertical force of 64 kN. Assuming a static coefficient of friction of 0.7, sliding will occur for horizontal forces greater than 88 kN. Therefore, the buttress will fail by overturning before sliding.

The adjacent structure to the North provides additional support to the North wall, and therefore the North wall can be assumed to remain vertical. The South wall is currently leaning by 0.4°, and the analysis here will assume that the South wall will continue to lean further, thereby endangering the structure and possibly leading to collapse.

The analysis of the arch on spreading supports and the leaning buttress is combined and presented in figure 9. The South buttress is assumed to lean progressively up to collapse, increasing the span of the arch; while the North buttress remains vertical. The program ArchLean has been adapted for the problem of only one leaning buttress, and has determined that collapse will occur when the South buttress leans by slightly more than 2.0°. At this point, the thrust of the arch exceeds the capacity of the South buttress and the arch will collapse due to the failure of the buttress. Thus, the failure state is a “weak-buttress” mode (Ochsendorf, 2002). In the current state, with the South buttress leaning by 0.4°, the thrust of the arch has increased to 41 kN, from its initial value of 39 kN. The collapse state of the church at Goa due to the leaning of the South buttress is illustrated in figure 9.

The relationship between the horizontal thrust in the structure and the leaning of the South buttress is illustrated in figure 10. The current state of the structure can be compared to the final collapse state due to the thrust of the arch exceeding the capacity of the leaning South buttress. For comparison, another plot is given for the same structure but with the actual arch replaced by an arch of smaller thickness ratio (t/R =0.05). In this case, analysis shows that the arch would collapse well before the thrust exceeds the capacity of the buttress and that the failure mode would be of the “strong-buttress” kind. This section has described the collapse condition for the church at Goa due to the outward leaning of the South buttress. The following sections apply measures of safety to the church, as well as a general procedure for assessing the safety of buttressed arches.

Load factor for collapse

For an arch supported on buttresses, the most obvious measure of safety is a simple load factor applied to the thrust of the arch. The load factor is equal to the horizontal thrust capacity of the leaning buttress H_ϕ divided by the thrust of the arch, or

$$SF_{load} = \frac{H_\phi}{H}. \qquad [1]$$

For the church at Goa as originally built, the vertical buttress has a thrust capacity of 69 kN, and the arch provides a minimum thrust of 39 kN, so the initial load factor of safety against collapse is 1.8. As the buttress has increased its lean over several centuries, the load factor has reduced. This can be appreciated from figure 10, where the buttress capacity is decreasing and the applied thrust is increasing. For the church in its present state with a lean of 0.4°, the buttress capacity is 65 kN and the applied thrust is approximately 41 kN due to the deformation of the arch. Thus, the load factor of safety has been reduced from 1.8 to 1.6 due to the current lean of the buttress. If the lean of the buttress were to increase steadily to 2°, the thrust of the arch and the carrying capacity of the buttress would be equal at 52 kN per metre, and the arch would collapse. At the collapse state, when the thrust of the arch equals the capacity of the buttress, the load factor is 1.0 (i.e. there is no reserve of safety).
The load factor measurement of safety is not valid in the event of a "strong-buttress" failure, in which the arch collapses first. This can be illustrated for the Goa church by assuming a smaller thickness ratio for the arch. Thus, if the thickness ratio were 0.05 instead of its actual value of 0.10, the arch would collapse before the buttress fails (as shown by the lower curve in **figure 10**). In this case, the thrust capacity of the buttress would still be 1.6 times greater than the thrust from the arch at the point when the arch collapses. Thus, the load factor for the thrust of the arch is only rationally applicable to a "weak-buttress" failure, in which the thrust of the deformed arch can exceed the capacity of the buttress. For the actual church at Goa, the arch could exert a thrust as high as 88 kN before collapsing due to spreading supports, which is greater than the thrust capacity of the buttress. Therefore collapse will occur due to a "weak-buttress" failure, and the load factor of safety of 1.6 is a valid measure of the safety against collapse.

Pressure-point factor

The pressure-point factor for buttresses is an additional measure of the influence of the horizontal overturning force (Ochsendorf, 2002). This measure considers the reaction point at the critical section of the buttress and investigates the location of this reaction point as the horizontal force increases and as the buttress leans outwards. The pressure-point co-ordinate η is measured from the outer edge of the buttress about which overturning would occur, and can be compared to the initial reaction point co-ordinate h_o if the buttress did not support any horizontal thrust. For the church at Goa, the hypothetical reaction at the base of the South buttress for zero horizontal force occurs at h_o =0.53. For a given lean and applied thrust, the reaction point η can be used to define the safety as:

$$SF_{pressurepoint} = \frac{\eta_o}{\eta_o - \eta} \qquad [2]$$

When the buttress is vertical (ϕ =0°) and the initial horizontal thrust of the arch is applied (H_a =39 kN), the reaction point at the base of the Goa buttress is η = 0.35, which lies within the middle third of the buttress. In its current state of leaning, the horizontal thrust of the arch has increased to 41 kN and the reaction point at the base has moved to η =0.29. This reaction point is computed for the leaning buttress by assuming that cracking has begun and that some of the buttress is no longer effective, since the reaction point is outside of the middle third. The pressure-point factor is summarised for the church at Goa in table 1.

Angle of lean, ϕ	**Thrust of arch, H**	**Reaction point, η**	**Pressure-point Factor**
0°	0 kN	0.53 (η_o)	∞
0°	39 kN	0.35	2.9
0.4°	41 kN	0.29	2.2
ϕ_u =2.0°	52 kN	0	1.0 (i.e. collapse)

Table 1. Pressure-point factor for the church at Goa.

In its undeformed state, the church at Goa had a pressure-point factor of 2.9, suggesting considerable safety. But with the slight lean of the South buttress to its current state of 0.4°, the reaction point at the base shifts from 0.35 to 0.29, decreasing the pressure-point safety factor to 2.2.

Just as with the load factor for collapse of the buttress, the pressure-point factor is only valid in the case of weak-buttress failure. If the arch collapses before the capacity of the buttress is reached, the reaction point at the base of the buttress cannot be used to assess the safety of the structure. For example, if the vault at Goa were half the thickness, with t/R =0.05, the vault would collapse before the capacity of the buttress is exceeded. In this case, the pressure-point safety factor would be 2.3, indicating that the buttress could withstand higher values of thrust; yet the arch would collapse before the capacity of the buttress is exceeded. Thus, the pressure-point safety factor is only valid when the horizontal thrust capacity of the buttress is exceeded and failure is governed by a "weak-buttress" collapse.

Safety assessment of an arch on leaning buttresses

The previous measures of safety have been concerned only with the value of the horizontal thrust applied to the buttress. However, in the event of a "strong-buttress" failure, the horizontal thrust capacity of the buttress is irrelevant (so long as it is greater than the maximum possible thrust from the arch). In this case, the safety factors presented above are irrelevant and positively misleading.

To determine the safety of an arch supported on leaning buttresses, the analyst must first investigate the arch and buttress independently and then together as a system. The most common structural problem for buttressed arches is the progressive outward leaning of the buttress. This dissertation has presented the general procedure to analyse this problem, although each individual structure will require a slightly different approach. To assess the safety of this structural system against collapse due to excessive leaning of the buttresses, the analyst should follow the approach summarised below in figure 11.

Thus, there is no single measure to determine the safety of an arch supported on leaning buttresses and the engineer must use judgement to assess the implications of additional buttress leaning. This will depend largely on the type of failure expected and the conditions of the particular structure. In general, there are two types of failure for an arch on leaning buttresses: strong-buttress and weak-buttress. In addition, it may sometimes be useful to think of an intermediate mode of failure between the two.

Each particular case requires an understanding of the current state of the structure and the implications of future movements on the safety of the structure.

This procedure for safety assessment can now be applied to the specific example of the church at Goa. Because failure occurs by a weak-buttress mode, the two measures of safety presented here can be applied to gain insight into the implications of increased buttress leaning. For comparison, the load factor and pressure-point factor are plotted in figure 12 for the church at Goa as the angle of buttress lean increases.

The load safety factor and pressure-point factor are both equal to 1.0 at the collapse state, though in general the load factor gives consistently lower values of safety. From this comparison, it is clear that the load safety factor is superior to the geometric safety factor. The pressure-point factor gives higher (i.e. more conservative) estimates of the safety and is much more difficult to calculate. Determining the geometric safety factor for different values of buttress leaning requires a computation of the progression of cracking and the movement of the reaction point up until the collapse state. The load safety factor is a very simple calculation, with immediate physical meaning in relation to the failure of the buttress. To assess the safety of the buttress against horizontal loads, the load factor of safety is the preferred method. However, the pressure-point factor can be used to assess serviceability issues for the structure, such as the danger of a fracture in the buttress due to the eccentric loading on the buttress.

For the church at Goa, the load factor suggests a level of safety of approximately 1.6. This factor is derived for the general structure in its current state, acting under only its own weight. Furthermore, this analysis did not consider the reduced weight of the buttress due to the presence of doorways and other openings in the masonry wall. Finally, this analysis assumes that the North buttress remains vertical and does not lean as the thrust increases. If the North buttress and South buttress were to lean apart at the same angle, then the arch would collapse for a smaller angle of buttress lean of approximately ϕ =1.2°. Thus, an analysis taking into account these factors would reduce the measure of safety further. The pressure-point factor suggests a higher level of safety, but the pressure-point is located at 0.29*b*, outside the middle third, suggesting that the walls may be fractured due to the eccentricity of the thrust at the base of the buttress. Is this a reasonable level of safety for this structure? The structure is clearly approaching a precarious state and additional leaning may lead to collapse.

Although the church has survived for over 400 years, the current state of leaning is approaching a dangerous level, and a factor of safety of 1.6 is not sufficient to ensure the long-term survival of the structure, and the addition of reversible steel tension ties to the church was likely justified.

Summary

This paper has examined the collapse state of an arch supported on buttresses, and several conclusions may be drawn. Traditional masonry structures can be analysed as rigid-block structures that may collapse due to applied loading, long-term displacements, or ground accelerations. Hitherto, researchers have focussed on understanding the load capacity of masonry structures, and have not sufficiently investigated the importance of displacements. For traditional masonry buildings, such as a masonry vault supported on buttresses, collapse is more likely to occur due to excessive displacements or ground accelerations than on account of an applied load.

In practice it is common to have an arch supported on leaning buttresses. The angle of lean may increase steadily over time, due primarily to foundation settlements; and the structure will eventually collapse. There are two general modes of collapse for this system: "weak-buttress" failure, in which the arch thrust will exceed the capacity of the buttress, and "strong-buttress" failure, in which the arch collapses before the thrust capacity of the buttress is exceeded. In both cases the arch will collapse and the buttress will remain standing.

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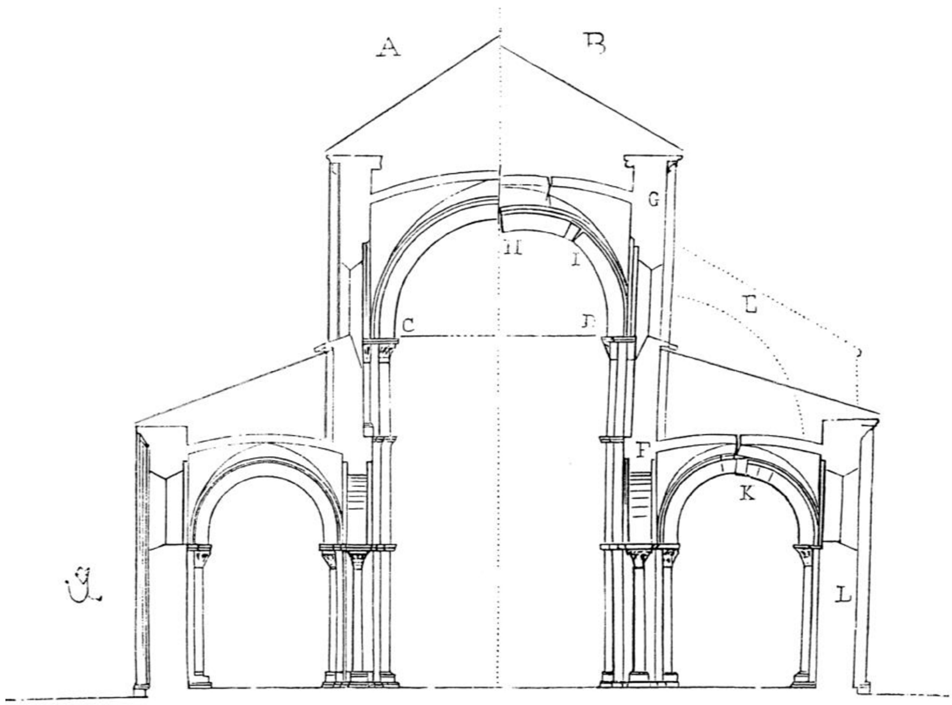

Fig. 1. Deformed state of the Church at Vézelay, France (Viollet-le-Duc, 1854). On the left as originally designed, and on the right as distorted.

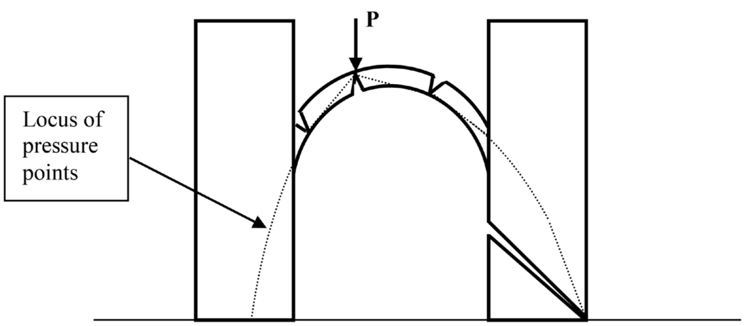

Fig. 2. Collapse of a buttressed arch due to a point load and the self-weight of the masonry.

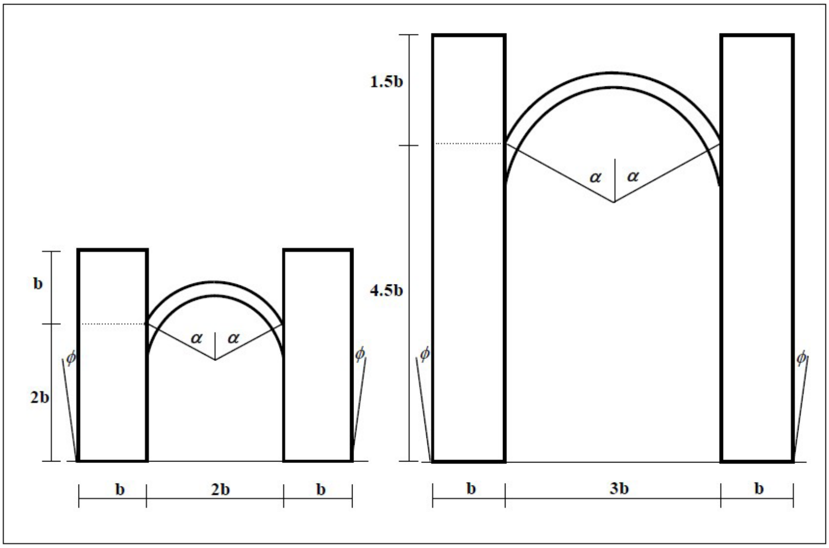

Fig. 3. a) Buttressed arch Case A b) Buttressed arch Case B

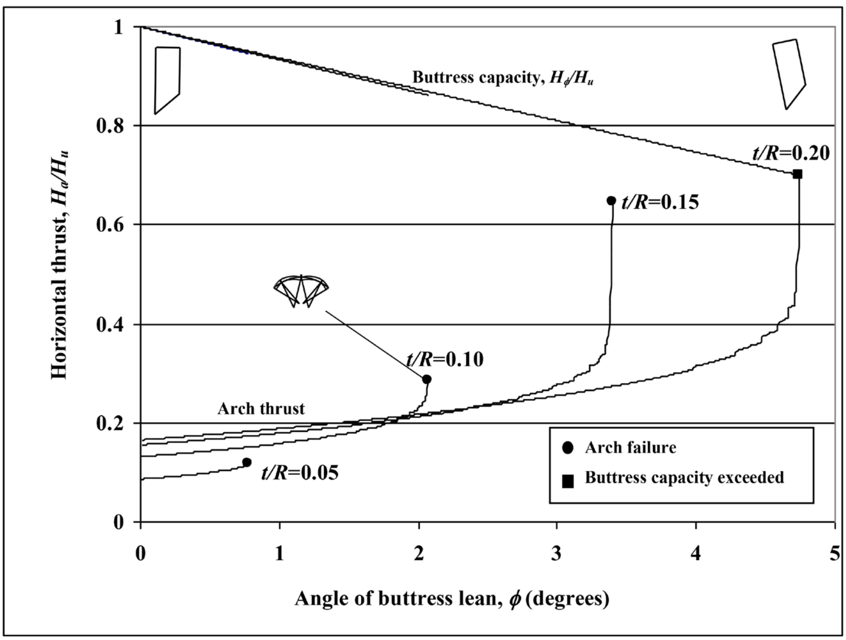

Fig. 4. Collapse state for buttressed arch Case A due to leaning buttresses (α=60°). The problem is analysed using the program ArchLean, for an arch constructed of 1° voussoirs in each case.

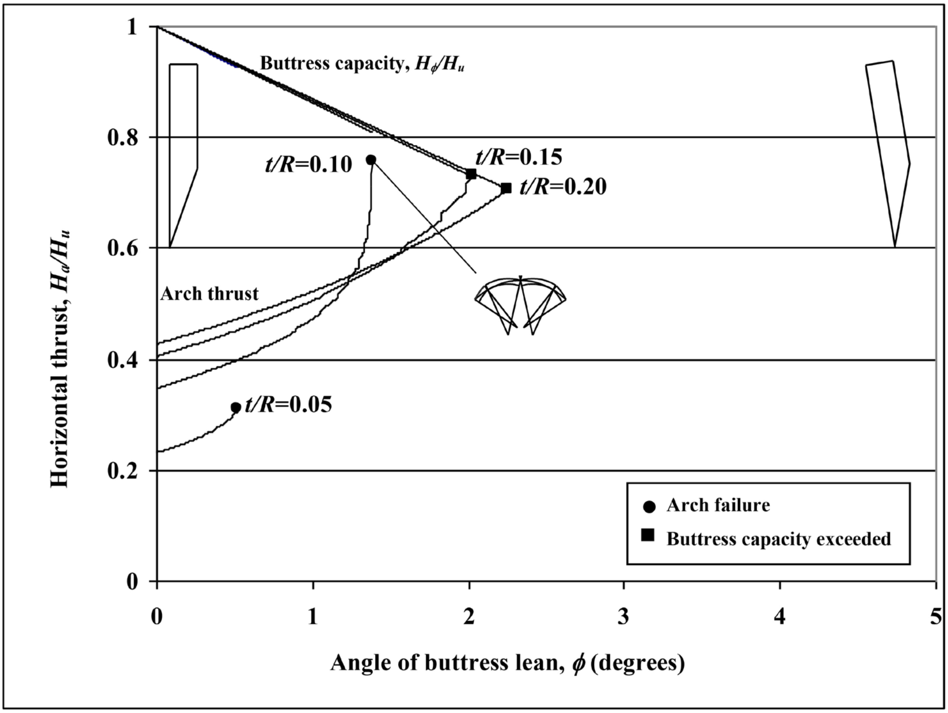

Fig. 5. Collapse state for buttressed arch Case B due to leaning buttresses (α=60°). The problem is analysed using the program ArchLean, for an arch constructed of 1° voussoirs in each case.

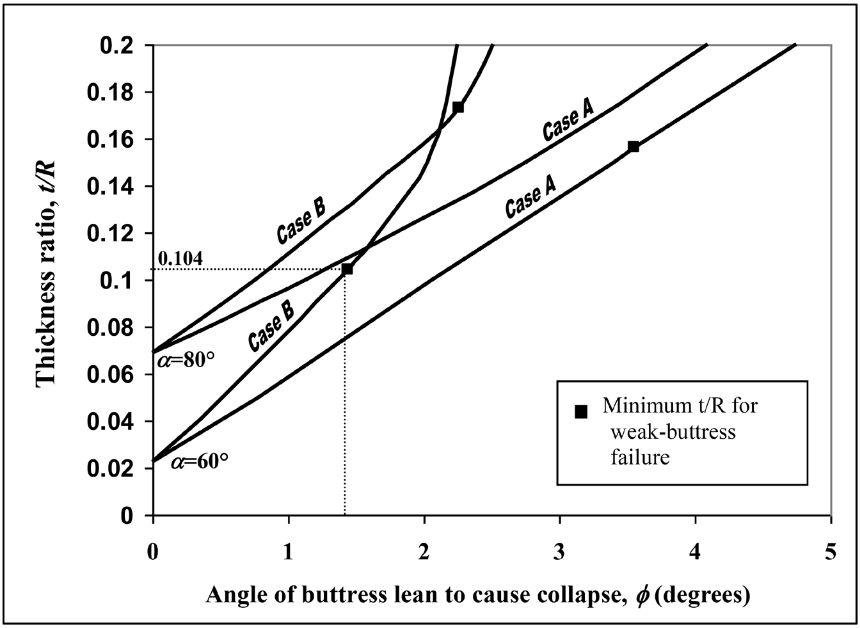

Fig. 6. Comparison of collapse states due to leaning buttresses for Case A and B.

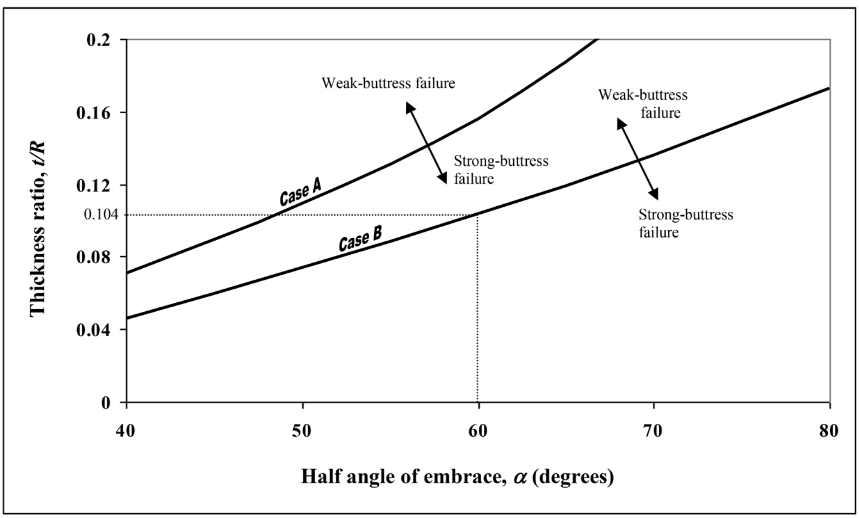

Fig. 7. Minimum thickness for the thrust of the deformed arch to exceed the capacity of the leaning buttress, i.e. threshold thickness between "weak-buttress" and "strong-buttress" failure. For example, an arch subtending 120° (α=60°) will fail by weak-buttress collapse when the thickness ratio is greater than t/R =0.104.

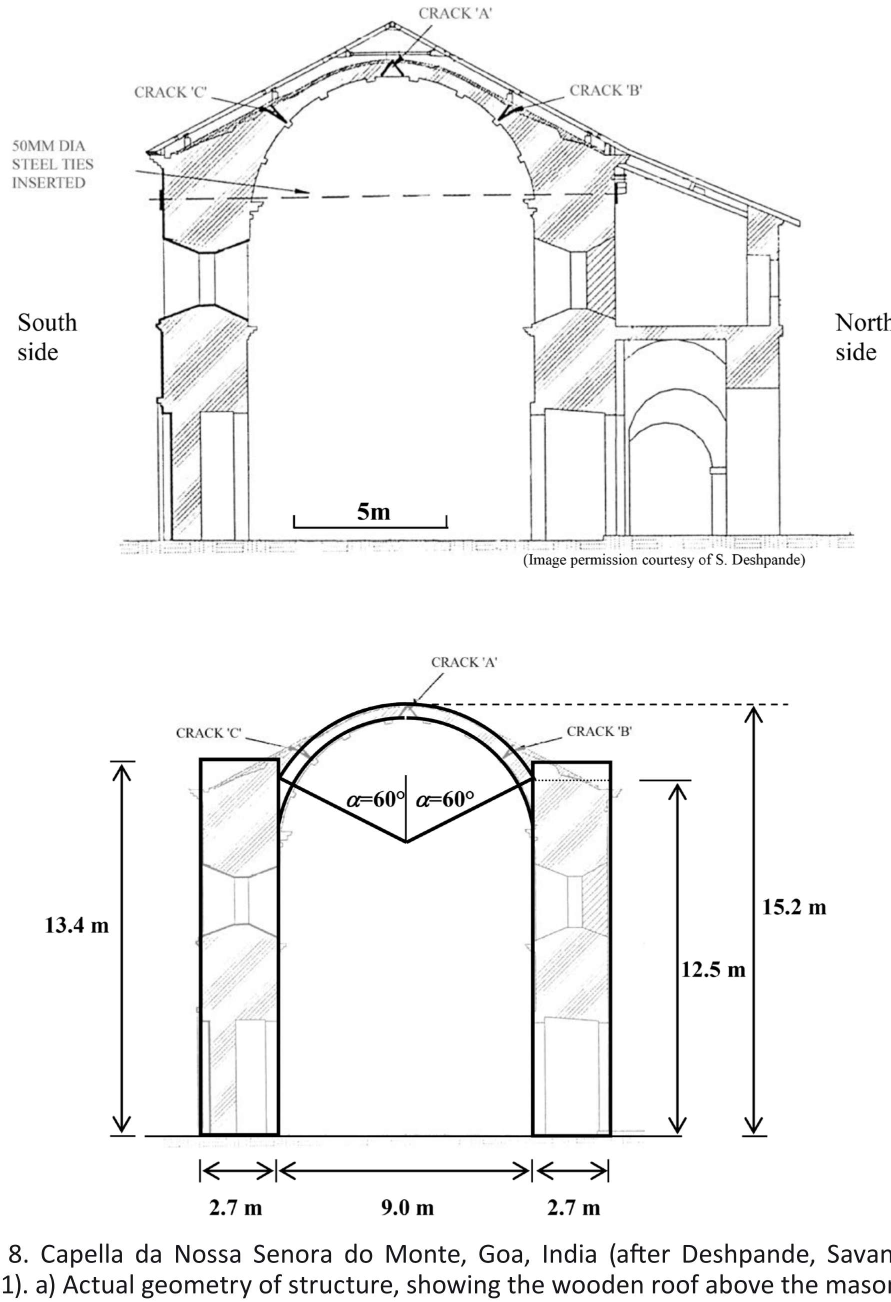

Fig. 8. Capella da Nossa Senora do Monte, Goa, India (after Deshpande, Savant, 2001). a) Actual geometry of structure, showing the wooden roof above the masonry vault and the location of cracking (hinges) in the vault. b) Idealised geometry of structure, assuming the buttressing walls are rectangular and the vault is circular of constant thickness.

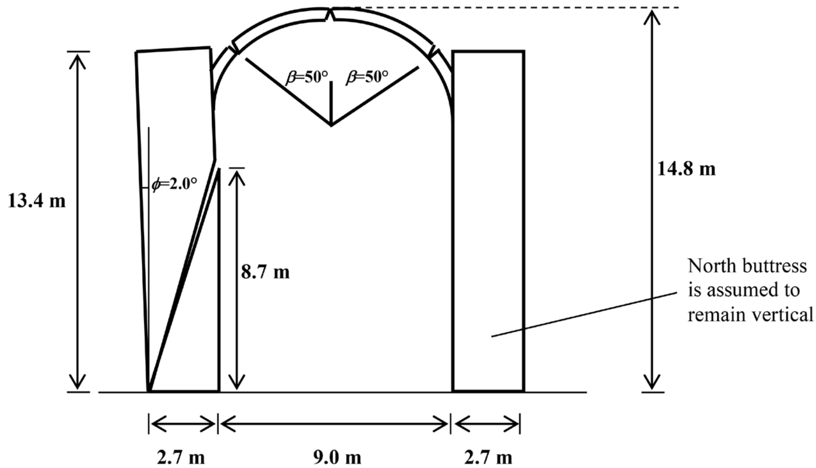

Fig. 9. Collapse state of the church at Goa due to leaning of the South buttress. When the buttress has leaned outward by 2°, the thrust from the distorted vault will exceed the thrust capacity of the buttress and the vault will collapse. At this point, the crown of the vault has descended by 0.4 m and the thrust of the arch will have increased from 41 kN to 52 kN.

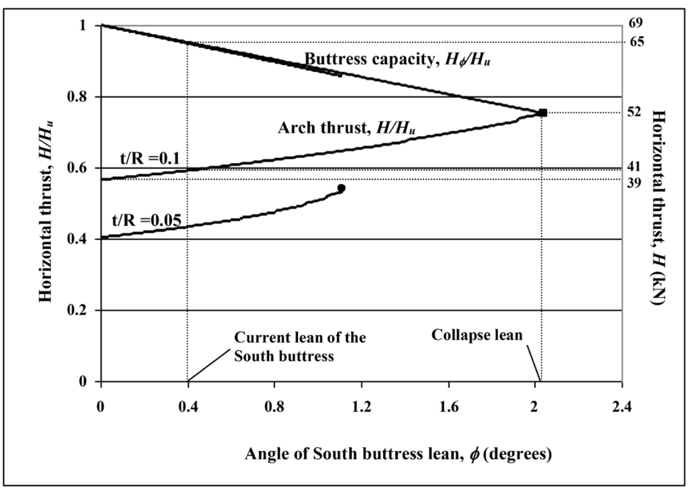

Fig. 10. Changing horizontal thrust as the South buttress leans for the church at Goa. Collapse occurs when the buttress leans by just over 2°, and the thrust of the deformed arch exceeds the capacity of the leaning buttress. If the arch were half the thickness (t/R =0.05), then the arch would collapse before exceeding the buttress capacity, at a buttress lean of 1.1° (shown by the lowest curve).

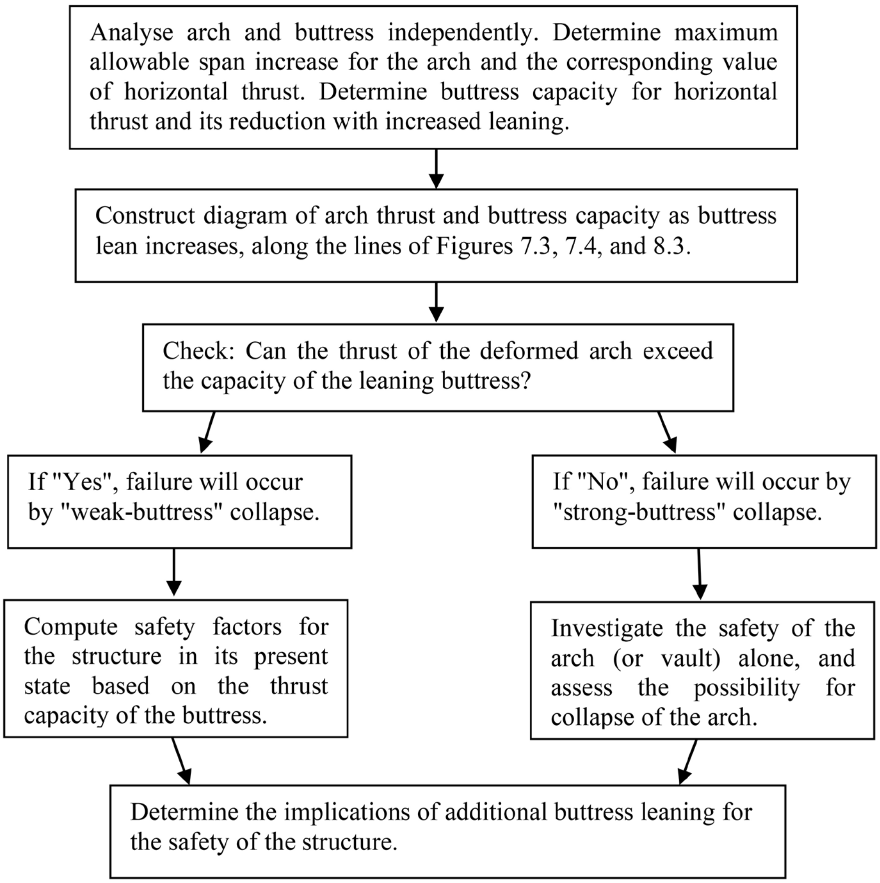

Fig. 11. Analysis procedure for assessing the safety of an arch supported on leaning buttresses.

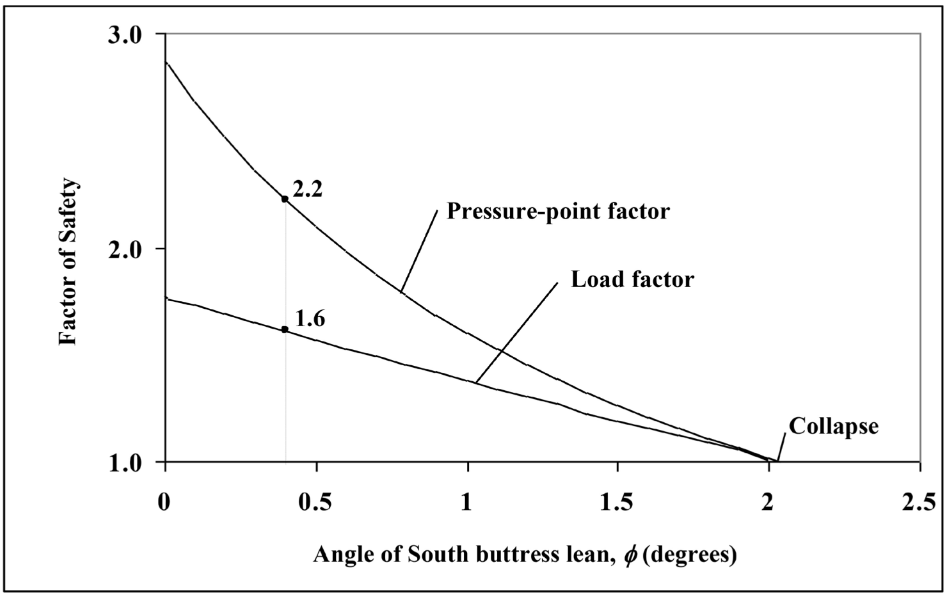

Fig. 12. Comparison of safety factors for the church at Goa.

Joaquín Francisco Antuña

Las estructuras de edificación de Eduardo Torroja Miret*

Origen del trabajo

En septiembre de 1999 se inauguró en Madrid una exposición conmemorativa del centenario del nacimiento de Eduardo Torroja organizada por el Centro de Estudios Históricos de Obras Públicas y Urbanismo (CEHOPU). Como parte del comité científico que preparó la muestra tuve acceso al archivo de la Oficina Técnica de Eduardo Torroja en donde se conservaban los expedientes de los proyectos en que trabajó desde 1924 hasta 1961. Después de su fallecimiento, la Oficina continuó en activo bajo la dirección de su hijo José Antonio y conservaron los documentos de la actividad técnica de Eduardo Torroja.
Unos años antes había solicitado el título de tesis para estudiar la obra de Eduardo Torroja, pero las necesidades laborales me obligaron a abandonar la dedicación a la tesis hasta que, en diciembre de 1998 me incorporé al comité científico que preparaba la muestra conmemorativa.
El interés en estudiar el trabajo de un reconocido autor en una disciplina tiene una componente didáctica fundamental para quien realiza el estudio, al menos, esa fue la motivación principal de mi elección del tema de la

***Agradecimientos**: A lo largo de los años en que se llevaron a cabo las diferentes tareas relacionadas con la investigación en torno a la actividad de Torroja he tenido que visitar bibliotecas, archivos, laboratorios, centros de investigación, obras y oficinas técnicas de arquitectos e ingenieros relacionados de alguna manera u otra con su trabajo. A todos con los que traté tengo que agradecer la colaboración amable, eficaz y desinteresada que me prestaron en cada ocasión y sin los que la tarea hubiese sido imposible.
Pero, sobre todo, debo un especial agradecimiento a tres personas. En primer lugar a Antonio de las Casas, gerente de CEHOPU en el tiempo de la exposición conmemorativa del centenario de Torroja, que confió en mi para participar en el comité científico que preparó la muestra y que me presentó a José Manuel Pedregal, antiguo colaborador de Torroja. Ambos me presentaron a José Antonio Torroja que me dio acceso a todos los documentos que conservaban en el archivo de la oficina técnica de su padre y con el que, desde entonces, mantuve contacto y gracias a quien una parte importante de las actividades que se incluyen en este texto fueron posibles. A la memoria de José Antonio Torroja, recientemente fallecido, quiero dedicar este texto.

tesis. Al estudiar la manera de trabajar de un técnico reconocido, se realiza una labor similar a la que se lleva a cabo en un sistema de formación de aprendices. Se puede seguir con detalle la manera de trabajar y deducir la manera en que se organiza la información, se analizan los datos y se toman las decisiones, aún sin la presencia del maestro.
La exposición conmemorativa del centenario se inauguró en septiembre de 1999 en Madrid y, en los años siguientes se presentó en varias ciudades como Ciudad Real, La Coruña, Granada, Lérida, Santander, Sevilla, Valencia y Vitoria, en España durante el año 2000. Entre los años 2000 a 2004 se presentó en varias ciudades por todo el mundo, como se puede ven en la página de CEHOPU.

Acciones derivadas del estudio del contenido del archivo

Al tiempo que consultaba los expedientes del archivo elaboré un catálogo en que se detallaba el contenido de cada expediente conservado. En los proyectos de edificación, en general, los expedientes consistían en la documentación necesaria para definir la estructura del edificio o, en algunos casos, todo lo relacionado con las instalaciones eléctrica, de acometidas y saneamiento. De cada proyecto se conservaban las memorias de cálculo, los pliegos de condiciones, las mediciones y presupuestos y los planos. En algunos casos también se conservan estudios previos y croquis preliminares de los proyectos. El conjunto estaba formado por casi novecientos expedientes de todo tipo de obras casi todas construidas, aunque también se conserva información de proyectos que no se llegaron a realizar.
La elaboración de un primer catálogo del contenido del archivo permitió identificar numerosos documentos inéditos, muchos de los cuales se mostraron en la exposición, así como en diversas publicaciones posteriores. Por otra parte, se pudo reconocer el valor de la información conservada y la utilidad que se seguiría de su difusión (Chías, Abad, 2005; Antuña, Casas-Gómez, 1999; Antuña, Pedregal, 2002; García-García, Antuña, 2006).
Gracias al trabajo realizado y a la iniciativa de José Antonio Torroja Cavanillas, que era el propietario del archivo, en 2002 se formalizó un convenio firmado entre el propio José Antonio Torroja y el Centro de Estudios y Experimentación de Obras Públicas (CEDEX), del Ministerio de Fomento, para que el contenido del archivo se depositase en el Centro de Documentación de CEHOPU.
El contenido del archivo donado se catalogó y ordenó y actualmente constituye el Archivo Eduardo Torroja depositado en CEHOPU. Una parte del

contenido del archivo está digitalizado y se puede consultar a través de su página web y el resto está disponible para su consulta en el centro. La cantidad de documentos digitalizados aumenta con las consultas que se van realizando con lo que es previsible que en un futuro próximo esté disponible el contenido completo (Centro de Estudios Históricos de Obras Públicas y Urbanismo, s.f.; Eito Brun, García, Sanz, 2011).

El hecho de poder acceder a la información contenida en el archivo ha permitido que se llevasen a cabo diversas iniciativas docentes, como el “proyecto Torroja” realizado en la Universidad Politécnica de Valencia, que consistía en utilizar proyectos realizados por Torroja como ejercicios en cursos de análisis de estructuras. Como resultado de esos cursos se realizaron varios trabajos de fin de máster y diversas publicaciones en que se analizaban las estructuras de proyectos de Torroja. En el caso de la Iglesia de Gandía, la información del archivo se pudo utilizar para la realización del proyecto de rehabilitación realizado (Payá-Zaforteza, García, 2013; Cabello, Paya-Zaforteza, Adam, 2014; Lozano-Galant, Paya-Zaforteza, 2017; Lozano-Galant, Payá-Zaforteza, 2011; Moragues *et al*., 2015; Oliver *et al*., 2016; Nuñez-Collado *et al*., 2013; Antuña, 2015).

La colaboración con arquitectos: Manuel Sánchez Arcas

Al estudiar en conjunto el contenido del archivo se pudo identificar con precisión la dedicación profesional de Torroja a lo largo de los años. Resultó evidente el cambio de orientación profesional que se produjo en 1929, coincidiendo con la dimisión de Rafael Benjumea como Ministro de Obras Públicas y la interrupción de la inversión en obra civil. Tarea a la que Torroja se había dedicado de manera principal.

Esta situación supuso un hecho fundamental que decidió la futura actividad profesional de Torroja. Ante la falta de expectativas realizando proyectos de obra civil, presentó una solicitud de trabajo para integrarse en el equipo de técnicos que realizaban el proyecto de la futura Ciudad Universitaria de Madrid. La solicitud fue admitida y, a partir de ese momento, y hasta 1961 realizó las estructuras de la mayor parte de los edificios de la Ciudad Universitaria. En los años siguientes hasta 1936, su principal actividad profesional se centró en proyectos de edificación. Su trabajo consistió en resolver los aspectos técnicos de las construcciones, colaborando con los arquitectos encargados de los proyectos. Uno de ellos fue Manuel Sánchez Arcas con quien realizó el proyecto del Hospital Clínico que comenzó a construirse en 1930. Dos años después, en 1932, realizaron el proyecto de la cúpula del mercado de Algeciras. En ambos edificios se incluyeron nuevas soluciones constructivas. Aunque el edificio del hospital era una estructura convencional incluía dos tipos de estructuras originiales. La cubierta de los quirófanos y las losas sin vigas de los voladizos de los balcones. Para comprobar ambas construcciones se realizaron ensayos, en el primer caso en modelo a escala y pruebas de carga en la obra construida, y en el segundo realizando y ensayando un módulo a escala real. En el caso de la cúpula de Algeciras, la única herramienta disponble para su comprobación era la construcción y ensayo de un modelo a escala. Las pruebas de carga en estructuras de hormigón se realizaban de manera habitual, pero es el primer caso en que tenemos noticia de ensayos en modelos a escala de estructuas de hormigón armado realizado en España.

Ambos técnicos, Arcas y Torroja, compartían la convicción de que la mejora de las condiciones técnicas de la construcción supondría una mejora de la sociedad. Esta coincidencia de intereses les llevó a participar en la fundación de una sociedad privada, "El Instituto Técnico de la Construcción" en 1934 cuyo principal objetivo era la difusión de las novedades técnicas en la construcción entre los técnicos y trabajadores (Chías, 1986; Instituto Técnico, 1934).

El hecho de reconocer la colaboración de Torroja en esos proyectos, permitió destacar la importancia de la solución de los aspectos técnicos en la definición de la arquitectura. En el monográfico sobre Manuel Sánchez Arcas publicado por la fundación COAM se incluyeron dos capítulos dedicados uno al Marcado de Algeciras y otro a la colaboración entre ambos técnicos (Antuña, 2003a; Antuña, 2003b). El libro obtuvo el "Premio a la Innovación. Fomento cultural 2005" otorgado por la Comunidad de Madrid.

La difusión de la colaboración entre ambos técnicos, así como el hecho de hacer que los documentos que definen la estructura de los edificios estén disponibles, permitió que se realizasen estudios como el realizado por Alba Lorente y sus colegas (Lorente de Diego, Martín-Gómez, Castro-Molina, 2018).

Por otra parte, el arquitecto Manuel Sánchez Arcas continuó su actividad profesional como arquitecto y urbanista en su exilio, primero en la Unión Soviética y, posteriormente en la República Democrática Alemana. Entre sus trabajos se encuentra una publicación sobre la construcción de cubiertas laminares de hormigón armado. En el texto repasa las construcciones realizadas organizándolas por los diferentes procedimientos constructivos utilizados. La obra se publicó en la RDA en 1961 y tiene interés porque, entre otras cosas, muestra la gran cantidad de procedimientos que se pusieron en práctica para mejorar la competitividad de la construcción de láminas. Junto con el profesor Fernando Castañón hemos preparado una traducción de la obra que está en proceso de publicación (Arcas, 1961; Arcas, 2022).

Las cubiertas del hipódromo de la Zarzuela

Durante la realización de la tesis y por encargo del CEDEX del Ministerio de Fomento realicé en el año 2000 un informe sobre el estado de las cubiertas de las tribunas del hipódromo de la Zarzuela. En esas fechas la actividad en el recinto del Hipódromo se había reducido casi totalmente. En los edificios no se realizaban labores de mantenimiento y las reparaciones y añadidos construidos en los años anteriores, en muchos casos contribuían a acelerar el proceso de deterioro. Se trataba de obras que se habían realizado sin ninguna atención al edificio existente y que, para su construcción se habían alterado los revestimientos y las impermeabilizaciones, dejándolos en una situación vulnerable. Esa situación, asociada a la falta de mantenimiento contribuyó a que aumentase el deterioro. Los añadidos y reparaciones que se habían hecho, no solo modificaban el aspecto del edificio degradándolo, sino que afectaban negativamente a su conservación.
Como resultado de esas actuaciones y la falta de mantenimiento, resultaban evidentes diferentes patologías. La situación era especialmente dramática en las cubiertas de las tribunas en que resultaba patente una fisuración generalizada en la cara inferior. Además, las manchas en toda la superficie mostraban la presencia de humedades permanentes en el interior de la lámina. Estas se concentraban en la proximidad del soporte central, en donde estaba situada la bajante por la que se evacuaba el agua de la cubierta. Por otra parte, se podían ver las reparaciones realizadas después de la guerra en que se habían rellenado las varias decenas de roturas que se produjeron. También resultaba evidente la reconstrucción de los extremos de los voladizos en los bordes de las cubiertas de cada edificio de tribunas. Aunque estas evidencias no eran un signo de deterioro, si mostraban la historia de lesiones y reparaciones que había sufrido el edificio.
Por otra parte, la impermeabilización de las cubierta estaba seriamente dañada. En grandes tramos faltaba la lámina impermeable y, en tramos amplios en los que se conservaba estaba despegada de la base de hormigón. De haberse mantenido esa situación durante varios años, la con-

servación de la cubierta hubiera quedado seriamente comprometida. El informe se entregó a Patrimonio Nacional, propietario del edificio que, años más tarde, decidió la rehabilitación de los edificios. La rehabilitación la llevó a cabo Junquera Arquitectos entre 2004 y 2015, asesorado en los aspectos estructurales por Leonardo Fernández Troyano de Carlos Fernández Casado S.L. (Castillo *et al*., 2011).
El estudio de la documentación existente en el archivo permitió identificar con detalle el proceso seguido en la concepción de la estructura de las cubiertas, desde la propuesta ganadora del concurso hasta la finalmente construida. Por una parte, se conservaban varios planos de la propuesta presentada al concurso en octubre de 1934, que consistía en una sucesión de láminas cilíndricas de generatrices perpendiculares a la dirección del edificio. Además, se conservan algunas propuestas intermedias y la solución finalmente construida definida en junio de 1935. La forma y el procedimiento constructivo elegidos permitían realizar la cubierta por tramos independientes que se podían realizar de manera más sencilla y económica que la solución original (Antuña, 2003c).
Uno de los objetivos de la tesis era situar la actividad de Torroja en relación con el estado de la técnica en cada momento, lo que llevó a estudiar el origen y evolución del uso del hormigón armado. Una consecuencia de este trabajo previo fue la participación en el comité científico de la exposición "Hormigón Armado. España: 1893-1936" realizada en 2010 en que se recopilarón las principales obras y autores de la construcción en hormigón armado realizadas en España en el periodo estudiado (Feduchi, Sáenz, 2010).
En los años siguientes participé en varios proyectos de intervención en estructuras de hormigón armado realizadas en el primer tercio del siglo XX. Una época que coincidía con la de los primeros proyectos realizados por Torroja.
La primera de ellas fue la colaboración en la rehabilitación del Teatro de La Comedia en Madrid. Un edificio de mitad del siglo XIX destruido en un incendio en 1915 y reconstruido ese mismo año con una estructura de hormigón armado por el arquitecto Luis Bellido. La propuesta de rehabilitación del edificio se eligió en un concurso restringido entre varios equipos de arquitectos entre los que resultó ganadora la propuesta presentada por los arquitectos Sebastián Araujo y Jaime Nadal, con los que colaboraba en el diseño de la solución estructural. El proyecto se realizó entre 2005 y 2010 y la obra concluyó en 2014. En la intervención se realizaron varias actuaciones en la estructura de hormigón armado, una de ellas consistió

en el añadido de una planta sobre el patio de butacas para situar una sala de ensayos y la otra incluir varios forjados más sobre el escenario con el fin de incluir la maquinaria escénica actual. La obra resultó premiada en la XIII Bienal Española de Arquitectura, en la sección “Patrimonio y Transformación” (Díaz-Urgorri, Santos, Moreno, 2016).
Años después participé en la rehabilitación del mercado de frutas y verduras de Legazpi en Madrid. Se trata de un edificio de hormigón armado realizado entre 1932 y 1935 que se encontraba en buenas condiciones, aunque con diversas patologías debidas al uso y la falta de mantenimiento. En ese proyecto colaboró Carmen Andrade, que había trabajado en la rehabilitación de las cubiertas del hipódromo de la Zarzuela y con quien hemos iniciado una línea de trabajo centrada en la evaluación de la resistencia de secciones de hormigón con armaduras afectadas con corrosión. Continuando con esta investigación se ha realizado un trabajo fin de grado en la Escuela Técnica Superior de Arquitectura de Madrid y ambos codirigimos una tesis doctoral en el programa de Doctorado en Estructuras de Edificación de la Escuela Internacional de Doctorado de la Universidad Politécnica de Madrid con el objetivo de desarrollar un método no destructivo de reconocer el grado de corrosión de las armaduras. Asimismo se está estudiando en qué medida se modifican las propiedades mecánicas del acero en función del grado de corrosión, estudiando aceros de las propias obras y que se deterioraron en situaciones de servicio a lo largo de los años y no mediante procedimientos acelerados de corrosión (Antuña, Andrade, 2020).

La cerámica armada y los depósitos elevados

El estudio de los expedientes del archivo muestra que Torroja realizó una investigación permanente orientada a obtener procedimientos constructivos eficientes, planteando alternativas específicas para cada tipo de problema. Una de las técnicas que exploró en varias ocasiones fueron las posibilidades de la fábrica de ladrillo para construir estructuras superficiales (Oschendrof, Antuña, 2003; Antuña, 2005; Antuña, 2010).
La técnica de la bóveda tabicada, habitual en España en la primera mitad del siglo XX, permite construir superficies de diferentes formas sin necesidad de encofrados. Asociada con el hormigón armado, la fábrica de ladrillo permite reducir el encofrado necesario, e incluso eliminar su necesidad y, con ello, el coste de construcción. El armado asegura la resistencia a tracción de la fábrica. De hecho, una de las primeras construcciones en

hormigón armado realizadas en España fue un depósito de agua que respondía a una patente de Anonio Maciá Llusiá y consistía en un muro de ladrillo que servía de encofrado a una pared de hormigón armado que entre ambos materiales apenas tenía 100 mm de espesor.
En la primera mitad del siglo XX se construyeron depósitos en hormigón armado y para evitar la fisuración del hormigón y garantizar la estanquidad, se limitaba la tensión de las armaduras. El pretensado sustituyó al hormigón armado como técnica para realizar depósitos. Por ello, cuanto Torroja recibió el encargo de modificar el proyecto de un depósito que estaba en construcción en la localidad Marroquí de Fedala, cerca de Casablanca, Torroja realizó una cuba con forma de hiperboloide de revolución utilizando el sistema de pretensado patentado por Ricardo Barredo.
En una comunicación de la asociación nacional del pretensado Torroja calificó esa obra como un fracaso de su técnica de pretensado. Explicaba que consideraba que el proyecto era un fracaso porque había utilizado un sistema patentado y, que en ese caso particular, no había sido capaz de desarrollar un sistema alternativo al empleo de un sistema convencional de pretensado. En los años siguientes desarrolló varios procedimientos constructivos para obtener paredes de depósitos pretensadas sin tener que recurrir al empleo de los sistemas de pretensado registrados.
Desarrolló dos propuestas que llegaron a incluirse en varios concursos a los que presentaron ofertas para la construcción de depósitos en Marruecos y en Francia. Uno de los procedimientos permitía construir depósitos elevados con forma de tronco de cono invertido. Para ello la pared se dividía en varios tramos por juntas con la dirección de las generatrices haciendo que la forma de cada tramo tuviese un a forma curva con una radio de curvatura mayor que el de la forma definitiva. La otra propuesta desarrollada combinaba muros de fábrica armada con muros de hormigón. El principio de este procedimiento estaba en la manera en que se construía la pared del depósito. La pared del deposito estaba formada por dos láminas, la exterior podía ser de hormigón o de fábrica de ladrillo en ambos casos armada y la interior de fabrica de ladrillo. Entre las dos se dejaba una cámara de unos quince centímetros de espesor. Una vez realizada la hoja exterior, se comenzaba a realizar el muro interior y, al mismo tiempo, se rellenaba de grava la cámara intermedia. Una vez concluido el muro con las tres láminas, exterior, cámara de grava e interior, se procedía a llenar el deposito de agua. El nivel de llenado era superior al que tendría el depósito en servicio. La presión hidrostática hacía que la armadura de la fábrica se alargase, llegando a producir la fisuración de la propia hoja exterior.

En esa situación, se procedía a inyectar mortero en la cámara dejada entre las dos hojas de fábrica y que se había rellenado con grava. La presión hidrostática debida al hormigón producía un aumento de la tracción en la armadura de la fábrica. El fraguado del hormigón y la disminución del nivel de agua hacía que se redujese la presión sobre la pared exterior y, con ello, el esfuerzo de tracción en la armadura de la fábrica disminuía con lo que tendía a acortarse. Sin embargo, este movimiento de acortamiento estaba impedido por el hormigón de la cámara que había fraguado. Entonces se producía una presión centrípeta que provocaba la compresión del hormigón de la cámara y de toda la pared del depósito.
Con este procedimiento se realizaron varios depósitos en la Junta de Energía Nuclear de Madrid en 1957. En la documentación del archivo Torroja hay referencias a expedientes de varios depósitos que se realizaron en la Junta, pero únicamente se conserva la descripción de uno de ellos, el deposito elevado de fábrica de ladrillo. También hay un expediente con el título “depósitos enterrados”, aunque sin contenido. El depósito de fábrica aún está en pie, aunque no en servicio. Con el fin de estudiar el estado en que se encontraba se solicitó un proyecto de investigación concedido en 2008. El proyecto permitió identificar, además del depósito de fábrica elevado, la existencia de tres depósitos enterrados construidos con un sistema similar, así como los documentos de esos proyectos que se conservaban en los archivos de el Centro de Investigaciones Energéticas y Mediambientales (CIEMAT), que es el organismo resultante de la transformación de la Junta de Energía Nuclear que encargó la realización de esos depósitos. A diferencia del depósito elevado, en el caso de los enterrados, la hoja exterior se hizo de hormigón armado.
Uno de los objetivos del proyecto era identificar el estado en que se encontraban las armaduras de las paredes de los depósitos, tanto su conservación como su nivel de tensión, para identificar si el procedimiento de pretensado había sido eficaz. En el depósito elevado de fábrica se pudo comprobar que el hormigón de la pared presentaba importantes coqueras, y no se pudo obtener ninguna muestra del tamaño necesario para estudiar la calidad del hormigón de la cámara.
La investigación coincidió con el proceso de desmantelamiento de las instalaciones nucleares que se estaba llevando a cabo en el centro. Entre las operaciones que se estaban realizando se incluía la demolición de los tres depósitos enterrados. Se pudieron identificar como los que se mencionaban en un expediente del archivo pero de los que no se conservaba ningún documento en el Archivo Eduardo Torroja. En el archivo del propio CIEMAT se encontró documentación que describía esos proyectos.

Gracias al proceso de demolición, se pudieron realizar varios ensayos destructivos en las armaduras de los depósitos, que permitieron comprobar que estaban sometidos a una tensión del orden de la mitad de su límite elástico. Un valor próximo a la tensión prevista para el depósito vacío, que era la situación de carga en que se encontraban en el instante del ensayo. Se demostraba con ello que el procedimiento ideado había sido eficaz.

Como resultado del proyecto de investigación concedido, se publicaron varias comunicaciones en congresos internacionales y una monografía en la que se describen las obras estudiadas y los resultados obtenidos (Antuña, 2008; Antuña, 2009a; Antuña, 2009b; Antuña, Carpintero, 2011).

Además de idear un sistema de pretensado utilizando la fábrica de ladrillo, Torroja exploró las posibilidades constructivas de las bóvedas tabicadas ligeramente armadas en las iglesias realizadas en la provincia de Lérida en torno a la construcción de la presa de Canelles. Se trata de construcciones modestas en las que realizó varias cubiertas utilizando bóvedas tabicadas para crear superficies de doble curvatura sin la necesidad de encofrado: la iglesia de Pont de Suert y la capilla de Sancti Spirit, hoy demolida. En la primera la nave tiene cinco tramos de cuatro metros de ancho, formados por arcos apuntados triarticulados. Cada arco consiste en dos partes de sección transversal de arco de circunferencia con radio variable con la altura. La superficie se genera al desplazar un arco de circunferencia de radio creciente a lo largo de otro arco de circunferencia. En el ábside la forma se obtiene por la rotación de una espiral sobre una directriz de arco de circunferencia. En ambos casos, la construcción se realiza sin encofrado. Con la ayuda de unas guías sencillas, la técnica de las bóvedas tabicadas permite su construcción sin encofrados (Antuña, 2005).

Se encuentra en preparación un monográfico sobre los depósitos elevados que Torroja realizó en Marruecos después de terminar el proyecto de Fedala. En el archivo se conservan los proyectos construidos y, además, diseños previos a los que finalmente se realizaron junto con propuestas que se elaboraron para concursos que no llegaron a realizarse. En esos documentos se puede seguir la evolución del sistema constructivo que se utilizó en los depósitos de la Junta de Energía Nuclear hasta su solución definitiva.

Entre las propuestas no construidas se conserva una para un depósito enterrado en Marrakes. Se trata de un depósito de planta hexagonal, con la cubierta soportada por una serie de pilares dispuestos dentro del depósito. La cubierta se dimensiona para soportar el peso de un relleno de un metro de tierra. Los soportes se sitúan en los vértices de una trama formada por rectas paralelas a las caras del hexágono. De este modo, se forman unos tramos de cubierta con forma de triángulo equilátero. La principal novedad de este depósito es una de las soluciones que se proponen como forjado de cubierta. Se trata de unas piezas prefabricadas de planta triangular, soportadas directamente en los vértices sobre los soportes. Estas piezas se construyen con una lámina de apenas cuatro centímetros de espesor y soportan el relleno previsto. La forma de los elementos prefabricados es la que permite equilibrar las acciones previstas, la carga de un metro de tierra, sin esfuerzos de flexión. Es la forma antifunicular de las carga para la disposición de los apoyos elegida en este caso, con los soportes en las esquinas de los módulos de planta triangular (Antuña, Orta, 2019).

Léxico de la construcción

Entre los documentos encontrados en el archivo se encontraron varios escritos con glosarios de términos habituales en teoría de estructuras y en construcción, con sus traducciones al francés, inglés y alemán. Una de las iniciativas que se puso en marcha en el Instituto Técnico de la Construcción y el Cemento fue la compilación de un léxico de la construcción. El trabajo estaba sin terminar al fallecer Eduardo Torroja y en 1962 se publicó la primera edición. En ella se recogen más de ocho mil términos de uso común en construcción y en teoría de estructuras y resistencia de materiales en aquellos años. Álvaro García Meseguer fue el encargado la edición. Entre la información que proporcionaba el léxico, estaba la indicación de los términos incluidos en el diccionario de la Real Academia Española (RAE).
La reedición del léxico fue uno de los trabajos abordados después de la presentación de la tesis y se publicó en 2009. Al realizarla se pudo comprobar que una parte importante de las definiciones de los términos que la RAE había incluido en el tiempo transcurrido desde la publicación del léxico, correspondían con la incluida en la primera edición del léxico (Antuña, 2009c).
Como resultado de esta reedición, en 2018 se firmó un convenio con entre la Fundación Eduardo Torroja y la Real Academia Española (RAE) con el fin de repasar los términos de construcción incluidos en el diccionario de la RAE[1]. El objetivo era revisar la terminología de la construcción incluida

[1] Ver <https://www.rae.es/noticia/convenio-para-revisar-el-lexico-del-ambito-de-la-construccion>.

en el Diccionario de la lengua española y, además poner a disposición de la RAE una lista de términos que no estaban incluidos y que la Academia podría añadir a su base de datos.

El trabajo de Torroja en el Laboratorio Central: Ensayos en modelos

En la preparación de la muestra conmemorativa del centenario de Torroja se pudo comprobar la variedad de actividades que desarrolló a lo largo de su carrera, además de su labor como proyectista de estructuras tanto para edificios como para obras públicas. Después de la guerra española ocupó la dirección de dos instituciones fundamentales en el desarrollo técnico en el sector de la construcción del tercer cuarto de siglo en España. Estas dos instituciones fueron el Instituto Técnico de la Construcción y el Cemento y el Laboratorio Central de Ensayos de Materiales de Construcción. El laboratorio se había fundado en 1899 y estaba dirigido por profesores de la Escuela de Ingenieros de Caminos. En 1939 Torroja se incorporó a la escuela como profesor y, a continuación, se le nombró director del Laboratorio. Como tal propuso una nueva organización del Laboratorio que incluía la construcción de un nuevo edificio en el que situar las diferentes secciones incluidas en la nueva organización.
Una de las novedades del nuevo laboratorio fue el crear un departamento de modelos reducidos, una técnica apenas desarrollada en España. Por esa razón, fue necesario formar al personal encargado tanto del diseño de los ensayos, la construcción de los modelos, la realización de los ensayos y la interpretación de los resultados. Consiguió formar un equipo técnico solvente que, bajo su dirección, realizó una intensa labor entre 1941 y 1961.
El trabajo de documentación para la exposición conmemorativa me permitió consultar los expedientes de los ensayos en modelos realizados en el laboratorio y presentar los resultados de ese trabajo en la publicación editada recientemente por Bill Addis (Antuña, 2019; Antuña, 2021b; Antuña, 2021a).
Los años en los que Torroja dirigió el Laboratorio Central coincidieron con el tiempo en que se extendió el uso de ensayos en modelos, convirtiéndolo en la herramienta necesaria para verificar la seguridad de estructuras de edificación complejas, o que utilizaban técnicas que no estuviesen suficientemente probadas. Para realizar los ensayos era necesario construir los modelos con un material adecuado, que permitiese movimientos de magnitud suficiente para que se pudiesen registrar con precisión, pero sin

que fuese necesario aplicar cargas elevadas. También era preciso desarrollar los dispositivos de recogida de datos y formar al personal necesario para todas las tareas necesarias. En los primeros años del laboratorio de modelos, se establecieron procesos de puesta en carga y se diseñaron dispositivos de medida que se utilizaron hasta 1961. A partir de esa fecha se introdujeron nuevos procedimientos de puesta en carga y equipos de recogida de datos más sofisticados. Sin embargo, esta renovación técnica y de procedimientos coincidió con una reducción en el número de ensayos realizados.
Coincidiendo con los ensayos de una obra del propio Torroja, la cubierta del canódromo de Madrid, se presentaron los primeros resultados de una investigación dirigida a desarrollar materiales adecuados para realizar modelos de manera rápida y económica. El objetivo final era poder utilizar los modelos no solo como una herramienta para comprobar una estructura con una forma y armados ya definidos, sino emplearlo también en las primeras fases del diseño para experimentar con diferentes alternativas de diseño. Esta búsqueda de materiales adecuados para la construcción de modelos se inició prácticamente con la puesta en marcha del laboratorio de modelos y se mantuvo hasta 1962. El objetivo era obtener materiales que se pudiese moldear con facilidad, de endurecimiento rápido y que, además, tuviese un módulo de rigidez longitudinal bajo y una resistencia también reducida. Por otra parte, debería ser un material económico. De ese modo se podrían fabricar en poco tiempo modelos de diferentes formas y someterlos a diferentes condiciones de carga. Los resultados se utilizarían para concretar la forma del proyecto definitivo. Se llegaron a construir modelos de compuestos de yeso para diferentes soluciones de la cubierta del canódromo. Esta línea de investigación se abandonó al tiempo que aumentaban las posibilidades de realizar modelos numéricos cada vez más elaborados y complejos.
El trabajo realizado con modelos se puede utilizar en la actualidad como herramienta docente. Y esa es una de las actividades que estamos desarrollando en el grupo de innovación educativa en el que participo en la Escuela Técnica Superior de Arquitectura de Madrid. La intención de una de las líneas de trabajo es recuperar los resultados de la investigación con modelos para su uso en la docencia. La manera de hacerlo es doble. Por una parte se utilizan los modelos construidos y ensayados, de los que se conocen las dimensiones y armados y los resultados de las pruebas realizadas, y se trata de explicar esos resultados utilizando modelos numéricos. Esta actividad se puede complementar, en algunos casos, con la toma

de datos en la propia obra construida, como ocurre con la citada obra del canódromo de Madrid. La otra forma de incorporar los resultados de la investigación con modelos es emplear los materiales que se llegaron a definir para que los estudiantes construyan modelos de estructuras que hayan diseñado ellos mismos.
Los modelos se han utilizado habitualmente como herramienta docente con varios objetivos: visualizar fenómenos mecánicos, comprobar experimentalmente soluciones obtenidas analíticamente o familiarizarse con el comportamiento mecánico de tipos de estructura habituales. En el grupo de investigación se realiza una actividad a partir de una idea original de Mariano Vázquez en la que el objetivo es lo que declara el título del propio taller: construir y romper estructuras. La principal novedad y originalidad es el planteamiento del taller. Los alumnos definen el problema estructural a resolver y las limitaciones que tiene que respetar de forma, material a utilizar, dimensiones, peso de la estructura y el tipo de carga que tiene que soportar, así como su magnitud. Con el objetivo establecido, cada grupo realiza un proyecto en el que define la forma y dimensiones de cada elemento de la estructura. A continuación construyen el proyecto realizado para, finalmente, someterlo a las cargas previstas. Si se alcanza el objetivo previsto, la carga se incrementa hasta la rotura. El proceso es tremendamente didáctico y permite a los alumnos familiarizarse con todas las fases del proyecto y construcción de una estructura y, en general, de un edificio (Antuña *et al*., 2019a; Antuña *et al*., 2019b; Antuña *et al*., 2019c).

Trabajo futuro

Una línea de trabajo en que estoy implicado actualmente tiene que ver con la actividad de Torroja en el Laboratorio central y con el uso de la información contenida en el archivo del propio laboratorio dirigida a dos objetivos. Por una parte, servir como apoyo a la docencia para mejorar y potenciar el uso de modelos físicos como apoyo a la docencia en las estructuras y, por otra, para servir de medio de estudio de la técnica de la construcción a lo largo del siglo XX.
El primer objetivo se centra en el estudio de los modelos ensayados para que sirvan de casos de estudio en trabajos de curso por una parte, y en la investigación en materiales para construcción de modelos para emplearlo en construir y ensayar estructuras en el aula.
Con el segundo objetivo se pretende identificar los materiales y técnicas utilizados en cada momento a lo largo del siglo a través de los resultados

de los ensayos realizados en el laboratorio. En el periodo entre 1900 y 1960 existen algo más de 20000 ensayos de los que se puede obtener una imagen precisa de las cualidades mecánicas de los materiales y sistemas empleados en la construcción. Este conocimiento servirá de ayuda a los técnicos que tengan que intervenir en los edificios existentes con el fin de garantizar una mayor eficiencia en los proyectos de intervención, rehabilitación, reparación o refuerzo.

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Experimental and numerical response of dry-joint masonry arches subject to large support displacements*

1. *Introduction*

Support displacements are one of the primary sources of damage for masonry arches, which are sensitive to small changes in the boundary conditions. The potential causes of support displacements are numerous, including foundation settlements, leaning of supporting pillars, soil heterogeneity, subsidence, and landslides. Although the movements produced by these phenomena are generally small in their instantaneous values, they can result in severe damage and even collapse if they increase significantly over time (Ochsendorf, 2006). Large support displacements are thus acknowledged as a major threat to the stability of masonry arches.

Considerable research effort has been made in the last two decades to assess the stability of masonry arches under large support displacements. Both experimental testing and analytical/numerical methods were used for this purpose.

As for the experimental tests, several authors tested small- or full-scale models of masonry arches under different configurations of support displacements. As described in Ferrero, Calderini, Roca (2022a), small-scale models were more widely used with respect to full-scale ones because they are less expensive and faster to be assembled, do not require significant building skills, and allow tests to be repeated several times. Furthermore, when dealing with masonry constructions, models at reduced scale can be confidently used to simulate full-scale structures as stability

* This work is based on the following research works: Ferrero *et al.*, 2021b; Ferrero, Calderini, Roca, 2022a; Ferrero, Calderini, Roca, 2022b; Ferrero *et al*., 2021a.

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is a matter of geometry rather than material failure (Heyman, 1995). Since the strength of the composing material is irrelevant, the small-scale models were built using a number of materials, including concrete, stainless steel, bricks, and wood, among the others. The voussoirs were assembled either with mortar or dry joints. The large majority of the experimental tests were carried out on arches subjected to horizontal and vertical support displacements, while little attention was devoted to inclined support displacements (Ferrero, Calderini, Roca, 2022a).
As for the analytical and numerical methods, a growing number of methods and procedures has been developed to investigate the response of masonry arches to large support displacements. The large majority of these methods modelled arches as rigid-no tension structures by adopting the three simplified assumptions introduced by Heyman (1966, 1995) to describe the behaviour of masonry materials: (I) infinite compressive strength, (II) no tensile strength, and (III) no sliding failure. Starting from these hypotheses, which provide the theoretical basis for the Limit Analysis theory, some authors proposed analytical and computational methods based on a standard application of the static and kinematic theorems of Limit Analysis (e.g., Block, Ciblac, Ochsendorf, 2006; Coccia, Di Carlo, Rinaldi, 2015; Di Carlo, Coccia, 2020; McInerney, DeJong, 2015; Ochsendorf, 2002, 2006; Romano, 2005; Romano, Ochsendorf, 2010; Zampieri *et al*., 2018a, 2018b; Zampieri, Simoncello, Pellegrino, 2018). Other authors developed procedures that, although adopting Heyman's assumptions on the behaviour of masonry structures, went beyond the standard application of the theorems of Limit Analysis (e.g., Ferrero *et al.*, 2021a; Galassi *et al.*, 2018; Galassi *et al.,* 2019; Galassi, Misseri, Rovero, 2021; Iannuzzo *et al.*, 2021; Portioli, Cascini, 2017). Finally, some authors used discrete element (DE) and finite element (FE) methods under Heyman's assumptions (e.g., Ayensa *et al.*, 2015; Ferrero *et al.*, 2021a, 2021b; Iannuzzo *et al.*, 2021; McInerney, DeJong, 2015; Zampieri *et al.*, 2018a). Similarly to what observed in the case of the experimental tests, also in the case of the analytical and numerical methods the largest attention was devoted to vertical and horizontal supports displacements, whereas inclined support displacements were little investigated (Ferrero *et al*., 2021a).
The large majority of the above-mentioned analytical and numerical methods were validated against the results from experimental tests on small- or full-scale models of arches with mortar or dry joints. As described in Ferrero *et al.* 2022b, when treating arches as rigid-no tension structures, the analytical and numerical procedures generally well captured the experi-

mental response in terms of opening of the initial hinges, hinge location, and collapse mechanisms, but they overestimated the experimental ultimate displacement capacity. This discrepancy was attributed to the imperfections of the physical models, which negatively affect stability and cause collapse to occur for a smaller support displacement than that predicted by perfect analytical/numerical models (Ochsendorf 2002, 2006; Galassi *et al.*, 2018).

In view of the foregoing, this work aims at (I) developing a deep understanding of the response of masonry arches to inclined support displacements and (II) proposing a numerical approach able to accurately simulate the behaviour exhibited by these structural elements when subjected to large support displacements. To accomplish these goals, a 1:10 small-scale model of a segmental dry-joint masonry arch was first tested to collapse under vertical, horizontal, and inclined support displacements. The experimental tests were then numerically simulated adopting a FE micro-modelling approach, in which the arch was schematized as an assemblage of voussoirs, very stiff and infinitely resistant in compression, interacting at no-tension friction interfaces. The arch response to large support displacements was assessed by means of nonlinear static analyses, in which support displacements were increased monotonically up to collapse. The proposed numerical approach was validated through comparison with the experimental results.

The present work is organized as follows. In Section 2, the experimental tests and results are presented. In Section 3, the adopted FE micro-modelling approach is described. In Section 4, the numerical predictions are compared with the experimental results. In Section 5, some conclusions are drawn.

2. *Experimental testing*

2.1. *Description of the physical model*

The experimental tests, presented in Ferrero, Calderini, Roca (2022a), Ferrero *et al.* (2021a) and Ferrero (2021), were performed on a 1:10 small-scale model of a segmental dry-joint masonry arch supported by two piers **[fig. 1a]**. The arch has an angle of embrace of 125°, a span length (*L*) of 533 mm, a radial thickness of 24 mm and a rise of 162 mm. It consists of 55 voussoirs with dimensions 24 x 12 x 120 mm^3 and slightly trapezoidal shape to compensate for the lack of mortar joints. As described more in

detail in Ferrero *et al.* (2021a, 2021b), the geometry of the arch is representative of the cross-section of a two-course brick barrel vault with a radial thickness of 0.24 m and an internal radius of 3 m.
The blocks of the mockup (both voussoirs and piers) were made of a bicomponent composite material (Plastoform PL, Prochima, 2009), which was obtained by mixing a mineral powder with an acrylic polymer in aqueous solution. The blocks were produced by pouring the mixture into special silicone moulds **[fig. 1c]** created from aluminium matrices shaped as the blocks **[fig. 1d]**. In the case of the voussoirs, the mixture was fluidified by adding a plasticizer (1% of the total weight) to facilitate the pouring.
The mockup was built as a dry-joint assemblage of bicomponent composite blocks because this manufacturing technique allowed the small-scale model to be coherent with Heyman's assumptions on the behaviour of masonry structures. The blocks had high compressive stress and stiffness with respect to the applied loads and, thus, could be considered rigid and infinitely resistant in compression. Furthermore, the dry joints had zero tensile strength and a high friction angle which caused sliding not to occur. The friction angle (μ), compressive strength (σ_c) and Young's modulus (E) of the blocks were respectively equal to 41.2° (corresponding to a friction coefficient of 0.7), 9.1 MPa, and 941 MPa, as measured experimentally (Ferrero *et al.*, 2022a). The density (ρ) of the bicomponent composite material was 1640 kg/m^3. The total weight of the mockup (including arch and supporting piers) was about 13.9 kg.
The arch was tested to collapse by moving the right support at a very low speed (maximum 0.04 mm/s) in vertical, horizontal, or inclined direction. Thirteen displacement directions were investigated by varying the angle α, which identifies the direction of the imposed displacement δ, between 0° and 90° **[fig. 1a]**. Since the angle α was measured from the vertical, α = 0° and α = 90° correspond to purely vertical and horizonal support displacements, respectively. The vertical and horizontal components of the imposed displacement δ, respectively named δ_z and δ_x **[fig. 1a]**, are expressed in a dimensionless form as δ_z/L and δ_x/L, where L is the arch span length.

2.2. *Results*

The arch response to large support displacements was assessed in terms of collapse mechanisms, evolution of the hinge configuration, and ultimate displacement capacity. The main conclusions drawn from the exper-

imental tests are reported in this section, while the reader is referred to Ferrero, Calderini, Roca (2022a) and Ferrero (2021a) for an in-depth analysis of the experimental results.
No matter the direction of the imposed support displacements, the arch initially opened three hinges, hereafter labelled A, B and C (beginning from the left support). For α between 0° and 75°, collapse was reached when a fourth hinge (hereafter indicated as D) opened at the left support. In the case of α = 90° only, due to the symmetry in geometry and displacement loading, a fifth hinge (E) also appeared at collapse at the right support.
Fig. 2 shows the collapse mechanisms obtained for four representative values of α (0°, 20°, 60°, 90°). Since hinges A, B and C may change position with the increase of support displacements, both their initial (A_0, B_0, C_0) and final (A_u, B_u, C_u) locations are indicated. The hinge position was found to depend on the direction of the support displacements. Based on the initial and final positions of hinges A, B and C, three modes of evolution of the hinge configuration with increasing support displacements were identified when varying α. The three modes can be described as follows:

- *Mode I* (for α between 0° and 15°, **fig. 2a**). The three initial hinges A, B and C follow the sequence I-E-E (from the left fixed support, where I = intrados and E = extrados) for every value of imposed displacement and do not generally move as support displacements increase. Collapse occurs by an asymmetrical four-hinge mechanism when hinge D opens at the left support at the extrados (sequence E-I-E-E).
- *Mode II* (for α equal to 20°, **fig. 2b**). The three initial hinges A, B and C are initially located according to the sequence I-E-E. As support displacements increase, hinge C closes at the extrados (right support) and opens at the intrados (right haunch). As a result, failure is governed by an asymmetrical four-hinge collapse mechanism with hinges located in the sequence E-I-E-I.
- *Mode III* (for α between 25° and 90°, **fig. 2c-d**). Hinges A, B and C alternate between the intrados and the extrados (sequence I-E-I) for every value of imposed displacement. The intrados hinges A and C generally move upwards towards the crown as the right support moves. For values of α up to 75° **[fig. 2c]**, collapse occurs by an asymmetrical four-hinge mechanism with hinges located according to the sequence E-I-E-I. For α equal to 90° **[fig. 2d]**, the arch collapses by an almost symmetrical five-hinge mechanism with two hinges (D and E) occurring at the springings (sequence E-I-E-I-E).

It is interesting to note that, for α between 20° and 30°, hinge C appeared in the form of minor openings distributed over consecutive joints. These minor and distributed openings (indicated in **fig. 2b** with dotted circles) were found to have the same effect as a hinge in the activation of the collapse mechanism (Ferrero et al, 2022a). As shown in **fig. 2b** for α = 20°, when hinge C occurred in the form of minor openings distributed, the thrust line at collapse (drawn on the arch deformed configuration by using graphic statics, see Heyman, 1992 and Huerta, 2011) did not touch the arch intrados at the right haunch, as expected in the case of a fully developed hinge, but it was almost tangent to it in correspondence to several consecutive joints. This behaviour differs from that expected for a rigid-no tension arch, which collapses when (at least) four fully developed hinges appear and the thrust line is tangent to the arch profile in at least four points (Heyman, 1995).

Fig. 3 shows the limit displacement domain of the tested arch, which was obtained by plotting the normalized vertical collapse displacement $\delta_{z,u}/L$ versus the normalized horizontal collapse displacement $\delta_{x,u}/L$ for every value of α. This domain, introduced for the first time by the authors in Ferrero *at al*. (2021a), indicates the combinations of vertical and horizontal displacements that the arch can withstand safely (points below the boundary of the domain) as well as those that cause collapse (boundary of the domain and points above it). Different trends in the variation of the collapse displacements with α can also be identified, which correspond to the different *modes* of evolution of the hinge configuration (for further details the reader is referred to Ferrero, Calderini, Roca, 2022a). Looking at **fig. 3**, it can also be seen that the arch had a significantly larger capacity to withstand vertical support displacements compared to horizontal support displacements. The vertical and horizontal collapse displacements obtained for α equal to 0° and 90° were equal to about 16.5% and 3.7% of the arch span length, respectively.

3. *Numerical modelling*

The numerical simulations were performed using the commercial FEM software DIANA FEA (TNO DIANA BV, 2014). A two-dimensional plane stress model of the tested arch, not including the supporting piers, was created in Midas FX+ Customized Pre/Post-processor for DIANA software (FX+ for DIANA, 2011). Following a micro-modelling approach, the arch was schematized as an assemblage of units, representing in size and shape

the real voussoirs, and zero-thickness interfaces, representing the dry joints. Further interface elements were placed at the arch springings to allow hinges to open at the supports. The edges of these interfaces were pinned to provide boundary conditions.
The mesh of the FE model was created using four-node quadrilateral isoparametric plane stress elements (Q8MEM) for the voussoirs and 2D four-node line interface elements (L8IF) for the interfaces. Following Ferrero *et al.*, (2021a, 2021b), a mesh size of 2 mm, corresponding to having twelve FEs along the arch radial thickness, was adopted.
The voussoirs were modelled as linear elastic elements with infinite compressive strength. The Young's modulus E = 941 MPa and the density ρ = 1640 kg/m^3 were taken equal to the values measured experimentally (see **Section 2**), while a Poisson's ratio of 0.2 was assumed.
All the nonlinearities were concentrated in the interfaces, which were modelled adopting a Coulomb friction model with cohesion and dilatancy angle set to zero. The friction coefficient μ was taken equal to 0.7, as measured experimentally. The friction model was extended with a gap criterion with zero tensile strength to allow hinges to open when tensile stresses arise. For further details about the Coulomb friction model and the gap criterion, the reader is referred to TNO DIANA BV (2014).
Special attention was devoted to the choice of the stiffness properties to be adopted for the interface elements, since they play a crucial role in the FE micro-modelling of dry-joint masonry arches (Ferrero *et al.*, 2021a, 2021b; Gaetani *et al.*, 2017; Lourenço *et al.*, 2010). The use of a Coulomb friction model requires to define two values of interface stiffness: the interface normal stiffness k_n and the interface tangential stiffness k_s. Since these properties were not measured experimentally, their effect on the numerical results was evaluated through a sensitivity analysis. Following the approach adopted in Ferrero *et al.* (2021a, 2021b), the interface normal stiffness k_n was varied within a range defined based on literature, whereas the interface tangential stiffness k_s was set equal to 0.5k_n for every value of k_n adopted. The effect of the interface normal stiffness on the arch response was first assessed for α = 0° (purely vertical displacements) by considering several values of k_n (**Section 4.1**). Subsequently, based on the results obtained, two reference values of k_n were chosen and adopted to analyse the arch behaviour when varying the direction of support displacements (**Section 4.2**).
Nonlinear static analyses were performed to simulate the experimental tests. First, self-weight was applied and then support displacements were

increased monotonically up to collapse. A regular Newton-Raphson iteration method was adopted in combination with a line search algorithm (TNO DIANA BV, 2014). To verify convergence, an energy-based convergence criterion with a tolerance value of 0.001 was assumed. Geometric nonlinearities were considered by adopting the Total Lagrange formulation available in DIANA (TNO DIANA BV, 2014).

4. *Comparison between experimental and numerical results*

4.1. *Sensitivity analysis for α = 0°*

In this section, the numerical and experimental results are compared for $\alpha = 0°$ in terms of collapse mechanism, hinge position at collapse and ultimate displacement capacity. The sensitivity of the FE predictions to the interface normal stiffness k_n is also evaluated by varying k_n between 0.1 and 100 N/mm^3, as proposed in Ferrero *et al.* (2021a, 2021b).

Fig. 4a-b-c-d reports the collapse mechanisms obtained for four representative values of k_n. Regardless of k_n, collapse occurred by an asymmetrical four-hinge mechanism with hinges located according to the sequence E-I-E-E. As observed in the experimental tests, hinges A, B and C were the first hinges to appear, whereas hinge D opened at collapse. As already commented in Ferrero *et al.* (2021a, 2021b), for small values of k_n, the hinges moved inward with respect to the intrados or extrados line of the arch due to the large interpenetration of adjacent blocks and, thus, generally appeared in the form of minor openings distributed over consecutive interfaces. In contrast, as k_n increased, they concentrated in fewer interfaces and moved towards the arch profile because of the smaller block interpenetration. For values of k_n equal or larger than 10 N/mm^3 **[fig. 4c-d]**, hinges A, B, C and D opened each in one interface, in full accordance with the experimental tests, and appeared at the edge line of the arch (compressive stresses are concentrated in only one FE of each interface).

Fig. 4e and **fig. 4f** respectively show the values of the collapse displacement and the position at collapse of hinges A, B and C as a function of k_n. The collapse displacement increased with increasing k_n until reaching a maximum constant value that was not affected by any further stiffness increase **[fig. 4e]**. The hinge position at collapse **[fig. 4f]** was the same no matter k_n for hinge C, whereas it changed with k_n for hinges A and B (note that the joints where hinges appear are numbered from left to right, being joint no.1 the one at the left support). As k_n increased, hinge A and C opened closer to the left and right support, respectively. However, in the range k_n = 48÷100 N/mm^3, hinges A and B were located at the same position regardless of k_n.

As can be seen from **fig. 4e** and **fig. 4f**, neither the ultimate displacement capacity nor the hinge location at collapse varied with the interface stiffness in the range k_n = 48÷100 N/mm^3. Consequently, for any k_n equal or larger than 48 N/mm^3, the interfaces can be considered as (almost) rigid and the arch can be treated as a rigid-no tension structure, as usually done in the literature when dealing with arches on moving supports (see **Section 1**). In Ferrero *et al.* (2021a), by comparing the FE results with the numerical predictions from a perfectly rigid block model (Portioli, Cascini, 2017), the authors actually proved that the value k_n = 48 N/mm^3 was representative of rigid interfaces.

Looking at **fig. 4e-f**, it can be seen that, if k_n is taken equal to 48 N/mm^3, the FE model significantly overestimates the experimental displacement capacity and does not accurately predict the experimental hinge position at collapse, even though it correctly simulates the experimental collapse mechanism. This outcome is in full accordance with the results from literature (see **Section 1**). In contrast, if k_n is reduced up to about 3 N/mm^3, the FE model is able to accurately predict the experimental collapse displacement and hinge position at collapse. Such a value of k_n is significantly smaller than the one used to simulate rigid (perfect) interfaces and, thus, represents rather deformable interfaces. In **Section 4.2**, k_n will be taken equal to 3 N/mm^3 and 48 N/mm^3 to perform further numerical analyses when varying α between 0° and 90°.

4.2. *Response for α = 0°÷90°: rigid vs deformable interface models*

In this section, the numerical predictions for both k_n = 3 N/mm^3 (deformable interfaces) and k_n = 48 N/mm^3 (rigid interfaces) are compared with the experimental results for all the directions of support displacements. The comparison is carried out in terms of limit displacement domain, collapse mechanisms and *modes* of evolution of the hinge configuration.

Fig. 5 shows the limit displacement domain of the arch obtained from both the experimental tests and numerical simulations. It is easy to see that the numerical model significantly overestimated the displacement capacity of the physical model for k_n = 48 N/mm^3, while it accurately predicted it for k_n = 3 N/mm^3. In the first case, the relative error between numerical and experimental collapse displacements ranged between about 22% and 53%, while in the second case it ranged between about -5% and 5% for every α, except for 10° and 90°, for which it was equal to -6.1% and 8.5%, respectively. Despite the differences in terms of ultimate displacement capacity obtained for k_n = 48 N/mm^3, for both values of k_n the numerical model was able to predict the overall qualitative trend exhibited by the experimental limit domain (i.e., how the vertical and horizontal collapse displacements varied with α).

Fig. 6 depicts the collapse mechanisms obtained for k_n = 3 N/mm^3 (left) and k_n = 48 N/mm^3 (right) for some representative values of α (0°, 20°, 25° and 90°). The initial and final locations of three initial hinges A, B and C is also reported. For both values of k_n, the FE model was able to capture the same collapse mechanisms obtained in the experiments (see **Section 2.2**). For k_n = 3 N/mm^3 **[fig. 6 on the left]**, the FE simulations also predicted the same *modes* of evolution of the hinge configuration identified in the tests when varying α. For k_n = 48 N/mm^3, the numerical results were in full accordance with the experimental ones for every α except 25°. In this latter case, although the numerical collapse mechanism was the same as the experimental one, *mode II*, instead of *mode III,* was predicted to occur **[fig. 6c on the right]**.

Despite the good agreement between experimental and numerical results obtained in terms of collapse mechanisms and *modes* of evolution of the hinge configuration for both k_n = 3 N/mm^3 and k_n = 48 N/mm^3, the numerical model was able to accurately predict the experimental hinge location only when adopting k_n = 3 N/mm^3. This can be easily seen from Table 1, which compares the predicted and experimental positions at collapse of hinges A, B and C. For k_n = 3 N/mm^3, the numerical hinge position was generally the same as the experimental one (or differed by a maximum of one voussoir). In contrast, when adopting k_n = 48 N/mm^3, the numerical and experimental locations did not generally match. For every α, hinge A was located closer to the left support and hinge B closer to the right support in the numerical model with respect to the physical one. In the case of hinge C, the experimental and numerical positions at collapse were in full accordance only for α between 0° and 15. In contrast, for α between 20° and 90°, hinge C was located closer to the right support in the numerical model compared to the physical one.

From **Table 1**, it can also be seen that the FE model was able to catch the opening of hinge C in the form of minor and distributed openings for α between 20° and 30°, as observed in the experiments, only when adopting k_n = 3 N/mm^3. In contrast, when using k_n = 48 N/mm^3, hinge C was predicted to occur in the form of a fully developed hinge.

Table 1. Position at collapse of hinges A, B and C obtained from experimental tests and FE analyses for k_n = 3 N/mm^3 and k_n = 48 N/mm^3 (I = intrados, E = extrados, m.d.o = minor distributed openings) (Ferrero, Calderini, Roca, 2022b).

α [°]	Joint no.								
	Hinge A			Hinge B			Hinge C		
	Exp	FEM (k_n = 3 N/mm^3)	FEM (k_n = 48 N/mm^3)	Exp	FEM (k_n = 3 N/mm^3)	FEM (k_n = 48 N/mm^3)	Exp	FEM (k_n = 3 N/mm^3)	FEM (k_n = 48 N/mm^3)
0	9-I	9-I	8-I	28-I	29-I	32-I	56-E	56-E	56-E
5	9-I	9-I	8-I	31-I	31-I	32-I	56-E	56-E	56-E
10	9-I	9-I	8-I	31-I	31-I	32-I	56-E	56-E	56-E
15	10-I	10-I	8-I	31-I	31-I	32-I	56-E	56-E	56-E
20	9-I	9-I	8-I	30-I	31-I	32-I	m.d.o.-I	m.d.o.-I	48-I
25	9-I	9-I	7-I	28-I	29-I	30-I	m.d.o.-I	m.d.o.-I	48-I
30	9-I	9-10-I	7-I	28-I	29-I	29-I	m.d.o.-I	m.d.o.-I	47-I
35	10-I	9-10-I	7-I	28-I	28-I	29-I	45-I	44-45-46-I	47-I
40	10-I	10-I	8-I	28-I	28-I	29-I	44-45-46-I	44-45-46-I	47-I
45	11-I	10-I	8-I	28-I	28-I	29-I	44-45-46-I	44-45-46-I	48-I
60	11-I	10-11-I	9-I	28-I	28-I	28-I	44-I	45-I	47-I
75	11-I	11-I	9-I	28-I	28-I	29-I	45-I	45-46-I	48-I
90	12-I	11-I	9-I	28-I	27-28-I	28-29-I	45-I	46-I	48-I

The results presented in this section show that, for all the directions of support displacements investigated, adopting k_n = 3 N/mm^3 provides a much better matching between numerical and experimental outcomes with respect to k_n = 48 N/mm^3. Since assuming k_n = 48 N/mm^3 means modelling rigid interfaces, it can be concluded that the joints of the physical model are not rigid but are characterized by a certain deformability. As described in Ferrero, Calderini, Roca (2022b), this deformability can be attributed to imperfections such as the roughness and not perfect coplanarity of the contact surfaces between adjacent voussoirs, which may result from the manufacturing process of the blocks.

5. *Conclusions*

This paper investigates the response of masonry arches to large support displacements through experimental tests and numerical simulations. The experimental tests were performed on a 1:10 small-scale model of a segmental arch, which was built as a dry-joint assemblage of bicomponent composite voussoirs and was tested to collapse under vertical, horizontal, and inclined support displacements. The numerical analyses were carried out by adopting a FE micro-modelling approach, in which the arch was schematized as an assemblage of voussoirs, very stiff and infinitely resistant in compression, connected by no-tension interfaces.

By analysing several combinations of vertical and horizontal support displacements, this paper provided a deep understanding of the behaviour exhibited by masonry arches when subjected to inclined support displacements. The direction of the support displacements was found to significantly affect the arch response in terms of collapse mechanism, evolution of the hinge configuration, and ultimate displacement capacity.

For all the directions of support displacements investigated, the comparison between experimental and numerical results was carried out using two different value of interface normal stiffness, which represented either rigid or deformable interfaces. In full accordance with the results obtained in the literature, the FE model with rigid interfaces, although accurately predicting the experimental collapse mechanisms, was found to significantly overestimate the experimental ultimate displacement capacity. This paper demonstrated that using deformable interfaces in the FE model could provide a very good matching between numerical and experimental results also in terms of ultimate displacement capacity. This outcome proved that the joints of the physical model were not fully rigid but characterized by a certain deformability, which was attributed to the presence of imperfections in the contact surfaces between adjacent voussoirs. Calibrating the interface stiffness based on the experimental results proved to be an effective strategy to accurately simulate the arch response to large support displacements and take into account the imperfections of the physical models.

Future research will be devoted to investigating the effect of the imperfections and deformability of the joints on the response of masonry arches to large support displacements. The study will be addressed to dry-joint masonry arches with different geometries as well as to masonry arches assembled with mortar joints.

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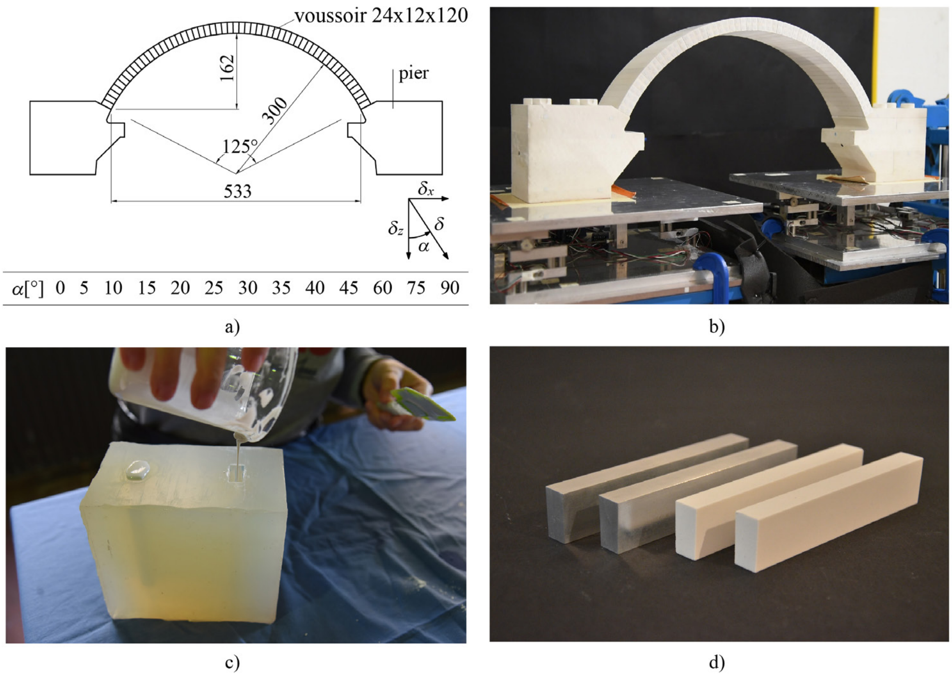

Fig. 1. a) Geometry of the mockup (dimensions in mm) and investigated displacement direction, b) view of the physical model, c) silicone mould used for the production of the arch voussoirs, d) alluminium matrices and bicomponent composite blocks (Ferrero, Calderini and Roca, 2022a-b).

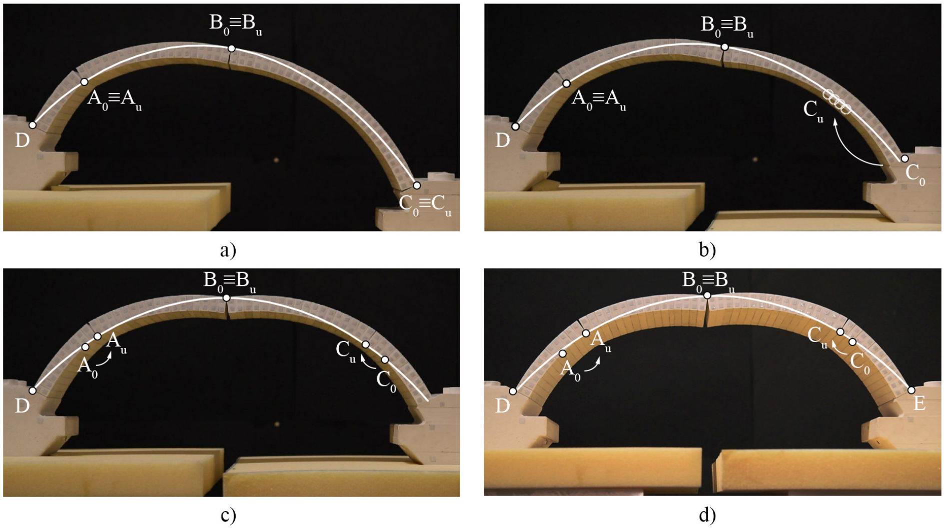

Fig. 2. Collapse mechanisms: a) $\alpha = 0°$, b) $\alpha = 20°$, c) $\alpha = 60°$, d) $\alpha = 90°$ (the initial and final locations of hinges A, B and C are indicated as A_0, B_0, C_0 and A_u, B_u, C_u, respectively) (Ferrero, Calderini and Roca, 2022b).

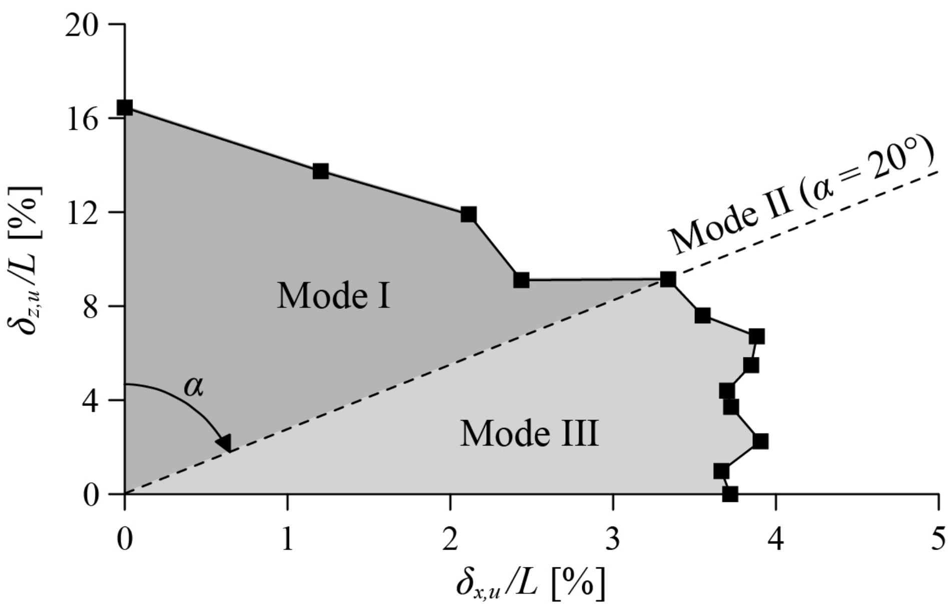

Fig. 3. Limit displacement domain of the tested arch (Ferrero, Calderini and Roca, 2022a).

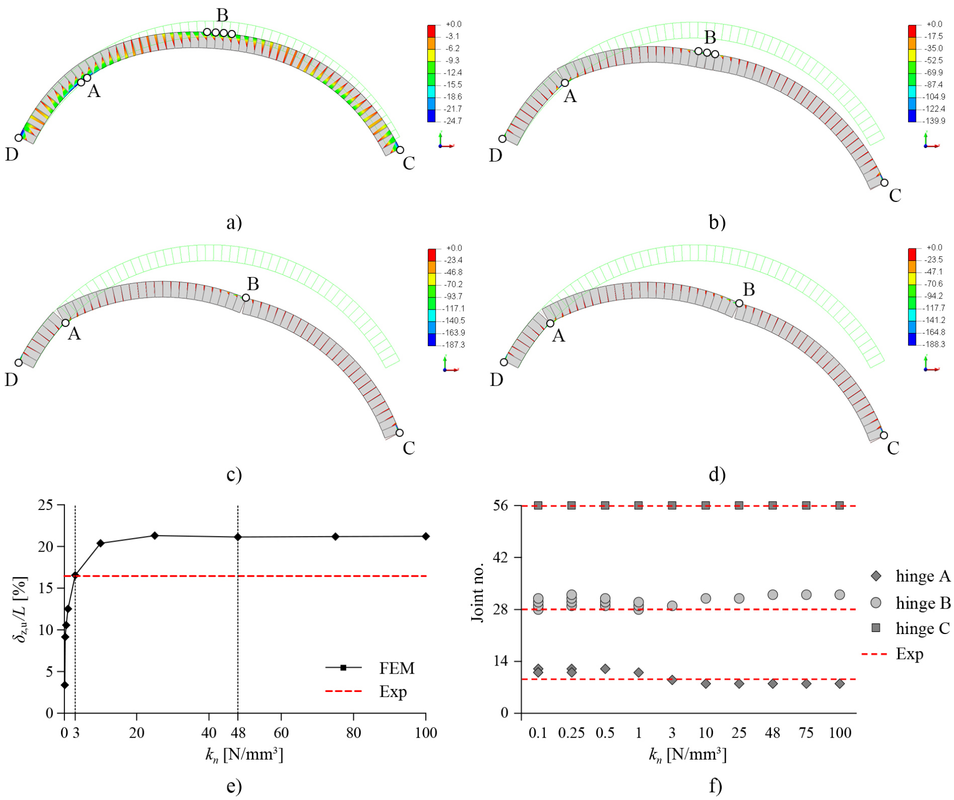

Fig. 4. Sensitivity analysis to the interface normal stiffness k_n: collapse mechanisms for (a) k_n = 0.1 N/mm³ ($\delta_{z,u}/L$ = 3.8%), (b) k_n = 1 N/mm³ ($\delta_{z,u}/L$ = 12.1%), (c) k_n = 10 N/mm³ ($\delta_{z,u}/L$ = 20.4%), (d) k_n = 100 N/mm³ ($\delta_{z,u}/L$ = 21.2%) (results in terms of compressive stresses in the interfaces); e) normalized collapse displacement $\delta_{z,u}/L$ vs. interface normal stiffness k_n; f) hinge position at collapse vs. interface normal stiffness k_n.. (Ferrero, Calderini and Roca, 2022b).

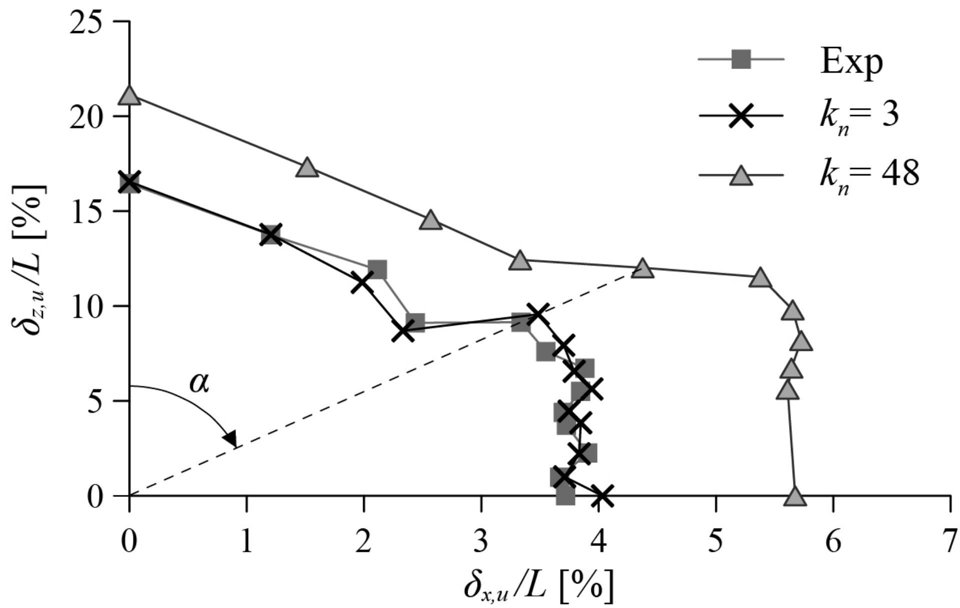

Fig. 5. Limit displacement domains of the tested arch obtained from experimental tests and FE analyses for both k_n = 3 N/mm^3 and k_n = 48 N/mm^3 (Ferrero, Calderini and Roca, 2022b).

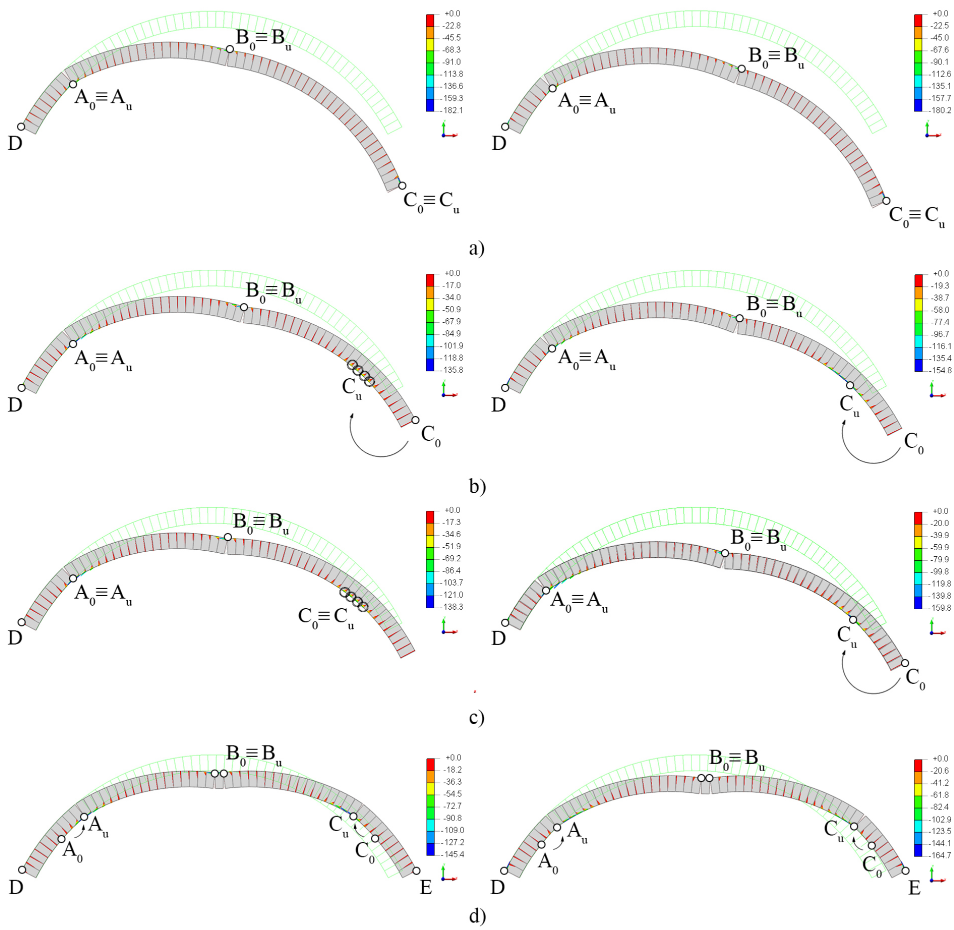

Fig. 6. Collapse mechanisms obtained from FE analyses for k_n = 3 N/mm^3 (left) and k_n = 48 N/mm^3 (right): a) α = 0°, b) α = 20°, c) α = 25°, d) α = 90° (Minor distributed openings are indicated with a dotted circle) (Ferrero, Calderini and Roca, 2022b).

Holger Eggemann

Simplified design of composite columns based on construction history and the development of design rules*

Construction history of composite columns

The construction history of composite columns can be divided into four periods:

1st: research started early in the beginning of the 20th century
2nd: regular use for highly loaded floors in factory buildings until the 1930s
3rd: followed by a period of oblivion until
4rth: a revival of research and application from the 1950s until today.

Although composite columns of concrete and steel were rarely used from the end of World War II until the early 1970s (Viest *et al*., 1997, 1.13), research had started a long time before, at the beginning of the 20th century. Combining of these materials had a number of motivations, steel columns were often encased in concrete to protect them from fire, while concrete columns were combined with structural steel as a reinforcement.

Until 1932, more than 1500 tested specimen in Europe and North America were reported by Emperger at the first IABSE Congress in Paris (1932), among those were 138 tests done by himself. Emperger complained about the lack of design rules for composite columns in Europe and mentioned the American "Standard Specifications for Concrete and Reinforced Concrete" of 1924, which gave explicit formulas for both composite columns and steel columns encased in concrete, a vital advantage for the application of composite columns during the 1920s and 1930s in tall buildings in Chicago. In Germany, it took until 1943 to apply composite columns in the German concrete regulations DIN 1045.

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After a period of oblivion, research in the field of composite construction was intensified during the 1950s and several design methods were developed. As a consequence, Klöppel´s proposal for concrete filled steel columns - first published (1935) - were taken into account for German steel regulations DIN 1050 in 1954. For today's Eurocode 4, the design method of Roik and his team was considered, developed in the 1970s (Roik *et al.*, 1975; 1976). This was taken as a basis for the proposed simplified design method.

Early examples in the United States

This paragraph refers to engineers who tried to find design formulas for columns combining structural steel and concrete and applied it in buildings. First documented design attempts in the United States were given by Talbot and Lord (1912) for steel columns reinforced with concrete and by Swain and Holmes (1915) for concrete filled steel pipes. Both teams tested more than 30 columns and used a straight line formula as common for steel columns at the time. While Talbot and Lord determined only the allowable stress of the steel section by a straight line formula, Swain and Holmes used this for both the steel and the concrete area, with a ratio of elasticity of steel to concrete of 9.6.

Also in 1912, William H. Burr made tests with steel columns filled with concrete, but gave no design formula, for tests were too few in number. Several years before, in 1908, he had applied such columns successfully in the construction of the Mc Graw Building in New York **[fig. 1]**, allowing an increased working load on the inner concrete:

> The use of the steel, in load-supporting condition, as a long column independent of the concrete, and at the same time forming a rigid banding member for the latter, with the consequent increase of permissible working load on the concrete, reduced the size of the columns in the basement and lower stories to dimensions quite consistent with the desired convenient and economical use of the clear floor space (Burr. 1908, p. 446).

Germany and Austria

In Germany, the first design formula for composite columns was given by Emperger (1913b, p. 32). The column type examined was a concrete column with a core of cast-iron and a strong horizontal reinforcement (Hohle

Gußeisensäule mit einem Mantel aus umschnürtem Beton), the later so called Emperger column **[fig. 2]**.

Emperger developed this column inspired by Melan arch bridges with embedded steel sections, called after its inventor Josef Melan, of which he had constructed the first ones in the United States during the 1890s. Emperger had this type of column patented in the German Reich in 1911 (Emperger, 1911) and applied it in the construction of the Ericsson factory building in Vienna in 1913 **[fig. 3]**. A similar type of column with mild steel encased instead of cast-iron was used in a telephone fabric in Budapest (Enyedi, 1931).

Emperger calculated the ultimate load P of a column as the sum of the strength of the materials (addition law - Additionsgesetz):

$$P = F_b\sigma_b + F_e\sigma_e + F_g\sigma_g \qquad [1]$$

where F is area, σ is stress, b is the index for concrete (Beton), e is the index for mild-steel reinforcement (Eisen) and g is the index for cast-iron (Gußeisen). This addition law is still valid today and used in the design of both concrete and composite columns.

Post WW II period and modern use of composite columns

After the second World War: «the introduction of lightweight fireproofing resulted in an almost complete elimination of composite columns from new buildings in the US.» (Viest *et al*., 1997, 1.13).

In Germany, because of the scarcity of steel as a building material, columns in building construction were mainly built of reinforced concrete with minimum reinforcement. In Europe, the use of composite columns was limited to special applications. Concrete-filled hollow sections were initially used in truss structures in overhead lines. A first large-scale application was 1964 for the overpasses of the freeway interchange at Almondsbury in England (Kerensky, Dallard, 1968).

Since the end of the 1960s, composite columns have been used again in high-rise construction in the United States (Viest *et al*., 1997, 1.16). Initially, following an idea by Fazlur R. Khan, the concrete portion of steel sections cast in concrete was used for the transfer of the wind loads, but today concrete-filled hollow sections are frequently used as mega-columns.

Six buildings in Seattle alone were constructed with concrete-filled hollow sections of large dimensions (diameter > 2 m), including the 243 m high office building Two Union Square (Viest *et al.*, 1997, 2.57). According to (Taranath, 1997), composite columns are frequently used as parts in lateral systems such as shear walls and mega frames.

To show the relevance of construction history, the composite column with a solid steel core should serve as an example. Such columns are used for particular high vertical loads when the dimensions are to remain small, e.g for columns in the façade. This type of column was used in the design of the Millenium Tower **[fig. 4]** in Vienna (Angerer *et al.*, 1999). In the author's opinion, this column can be regarded as the successor to the Emperger column.

Building regulations

ASCE Progress Reports 1910 - 1917

At the annual convention of the American Society of Civil Engineers on June 11th, 1903, a special committee was appointed:

> to take up the question of concrete and steel-concrete, and that such committee co-operate with the American Society for Testing Materials, and the American Railway Engineering and Maintenance of Way Association (ASCE, 1910, p. 431).

In 1904, the name of the committee was changed to “Special Committee on Concrete and Reinforced Concrete”. In its First Progress Report, recommendations were given for both concrete and reinforced concrete construction. In paragraph 9 concerning columns, it was said that:

> columns may be reinforced by means of longitudinal rods, by bands or hoops, by bands or hoops together with longitudinal bars, or by structural forms which in themselves are sufficiently rigid to act as columns (ASCE, 1910, p. 450).

For columns reinforced with structural steel, stresses were allowed 45% higher than for columns with longitudinal reinforcement only. In the discussion of the paper, several engineers remarked that the report had been written to fast and that allowed stresses had not been verified by test results. In the Second Progress Report, it was said that:

> Composite columns of structural steel and concrete in which the steel forms a column by itself, should be designed with caution. To classify this type as a concrete column reinforced with structural steel is hardly permissible, as the steel will generally take the greater part of the load. When this type of column is used, the concrete should not be relied on to tie the steel units together or to transmit stresses from one unit to another. The units should be adequately tied together by tie-plates or lattice bars, which, together with other details, such as splices, etc., should be designed in conformity with standard practice for structural steel. (ASCE, 1914, p. 421).

This paragraph can be seen as an attempt to reject composite columns from concrete construction and to pass it to the steel side of structural engineering. This paragraph was kept up in the Final Report (ASCE, 1917), but concerning reinforced concrete columns, a vital improvement was made; only columns with horizontal reinforcement were allowed and:

> It is recommended that the minimum size of columns to which the working stresses may be applied be 12 in. out to out. … Hooping is to be circular and the ends of bands must be united in such way as to develop their full strength (ASCE, 1917, p. 1133).

Nevertheless, composite columns were adopted again in the first American Standard Specifications No. 23 in 1920.

ACI Standard Specifications No. 23 - 1920

Design rules for two types of composite columns were given in the ACI Standard Specifications No. 23, “Standard Building Regulations for the Use of Reinforced Concrete”. First type were steel columns filled with concrete and encased in a shell of concrete, as used in the Mc Graw Building in New York, and second type were:

> Composite columns having a cast iron core or centre surrounded by concrete which is enclosed in a spiral of not less than one-half of 1 per cent of the core area, and with a pitch of not more than three inches, may be figured for a stress of 12,000 – 60 L/R, but not over 10,000 lb. per sq. in. on the cast iron section and not more than 25 per cent of the compressive strength specified in Section 40 on the concrete within the spiral or core. The diameter of the cast iron core shall not exceed one-half of the diameter of the spiral (ACI 23, 1920, p. 301).

Obviously, the latter one was the Emperger type of column, and was in common use until the 1930s in the construction of tall buildings in Chicago (ENR 1930, p. 278). This can be considered the first climax of composite column application. At the same time in Europe, few buildings were constructed with composite columns, due to the lack of design rules, as explained by Emperger (1932, p. 595).

German Standard DIN 1045 - 1943

During the 1930s, tests in Germany were made by Memmler, Bierett and Grüning (1934) on steel columns filled with conrete and by Gehler and Amos (1936) on concrete columns reinforced with structural steel. The latter ones led to the first German building regulations on composite columns. DIN 1045-1943, §27 d) treated concrete columns reinforced with structural steel. Column resistance was calculated by the addition law and a factor ω considering buckling. This paragraph existed until 1972, when the design of reinforced concrete columns was changed to ultimate load theory and officials decided that structural steel members encased in concrete should be designed to carry the loads alone without considering the strength of the concrete (Bonzel, Bub, Funk, 1972, p. 49).

German Standard VDE 0210 - 1953

This paragraph explains the post-war development of concrete filled hollow sections beginning with its use in overhead transmission line masts, e.g. in the construction of the Nufenen pass line in Switzerland, built in 1947. This system by "Motor-Columbus" was patented in Switzerland and several European countries (Girkmann, Königshofer, 1952, p. 320). The concrete filling allowed a notable saving of steel in post-war economy. In 1953, a paragraph on the design of such sections was added to German standard VDE 0210, "Vorschriften für den Bau von Starkstrom-Freileitungen" (Regulations for the construction of high-voltage overhead transmission lines).

For the first time, the term of an ideal or effective slenderness was used in the design of composite columns, considering the stiffness of both the concrete and the steel section together. The ideal or effective radius of gyration was calculated by:

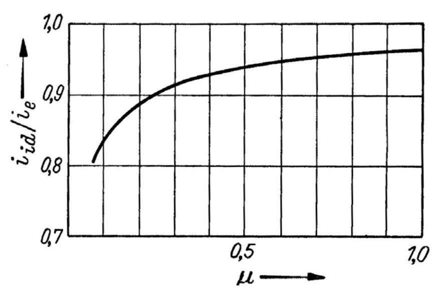
$$i_{id} = \sqrt{J_{id}/F_{id}}, \quad J_{id} = J_e + J_b / n \qquad [2]$$

where

J_e is the moment of inertia of the steel pipe
J_b is the moment of inertia of the concrete filling and
n is the relation of the moduli of elasticity of steel to concrete with a default value 10

This term, the ideal radius of gyration, explains the problem of buckling much more graphically to students than the modern non-dimensional slenderness. Therefore, the objective was to reintroduce it within the developed simplified design method.

Since construction was restricted to columns with a slenderness bigger than 50, Klöppel and Goder made tests with concrete filled sections in 1957 to extent the practicability of this design method to usual column sizes. They tested 54 filled and 45 pure steel pipes. In their abstract (Klöppel, Goder, 1957, p. 47) they compared the radius of gyration of the composite section with that of the bare steel section, and showed the relation in a diagram. This type of diagram turned out as a practical means in the developed simplified design method.



Relation of radii of gyration of composite to steel section
(Klöppel and Goder 1957, p. 47)

German Standard DIN 18806 - 1984

German Standard 18806 was the first building regulation in Germany dedicated exclusively to composite columns, for the first time covering three different types of composite columns with one design concept. Its design rules were based on the work of Roik, Bergmann, Bode and Wagenknecht (1975, 1976). The basic principles were kept up in Eurocode 4 method, see next paragraph.

Eurocode 4 - 1994

The simplified design method of Eurocode 4 is also based on the method developed at Bochum University by Roik and his team. The ultimate load of a composite column is given similar to Emperger´s Addition-Law by:

$$N_{pl,Rd} = A_a f_{yd} + A_c \alpha_c f_{cd} + A_s f_{sd} \qquad [3]$$

with f_{yd} the yield strength of structural steel, f_{cd} the compressive strength of concrete and f_{sd} the yield strength of reinforcement steel. Buckling safety is provided by a factor k, by which the ultimate load has to be multiplied. The factor k is given by European buckling curves of Eurocode 3. It has to be calculated as a function of the non-dimensional slenderness $\bar{\lambda}$. Roik based the design of composite columns on the design of steel columns, using the European buckling curves and the non-dimensional slenderness.

Table 1. Design of Composite Columns, Germany (Eggemann, 2003, p. 54).

Emperger, 1913	Hooped column with a core of cast-iron	$P = (F_b\sigma_b + F_e\sigma_e + F_g\sigma_g) / (4\ \omega)$ [kp] Addition-law
DIN 1045, 1943 Reinforced Concrete	Columns reinforced with structural steel	$P_{zul} = (K_b F_b + \sigma_S F_e) / (3\ \omega)$ [kp] Addition-law
DIN 1050, 1954 Steel Construction	Steel columns with a core of concrete	$S \leq \frac{\sigma_{zul}}{\omega_x}\left(F_e + 0{,}5\frac{W_{b28}}{\sigma_k}F_b\right) \leq 1{,}33\ F_e \frac{\sigma_{zul}}{\omega_x}$ [kp] Strength of concrete W_{b28} valued only 50%
VDE 0210, 1953 Construction of high-voltage overhead lines	Concrete filled steel pipes	$\sigma = \omega\ S / F_{id} \leq \sigma_{zul}$ [kp/cm²] Effective slenderness
DIN 18806 T1, 1984 Composite construction - Composite columns	Composite Columns	$N_{kr} = \kappa \cdot N_{pl}$ [kN] χ function of $\bar{\lambda}$
DIN 18800-5, 1990 Eurocode 4, 1994 Composite construction	Composite Columns	$N_{Rd} = (A_a f_{yd} + A_c \alpha_c f_{cd} + A_s f_{sd}) \cdot \chi$ [kN] χ function of $\bar{\lambda}$

Table 2. Design of Composite Columns, United States of America (Eggemann, 2003, p. 55).

Talbot and Lord, 1912	Concrete as reinforcement for structural steel columns	P = A (36500 - 155 l/r) [lb] Tetmajer-formula
Swain and Holmes, 1915	Concrete-filled pipe columns	$P = 1100\,(A_c + 9{,}6A_s)$ $P = (A_c + 9{,}6A_s)\,(1600 – 7\,L/r)$ [lb]
ASCE Reports, 1910-1917	Columns reinforced with structural steel	$P = f_c\,(A_C + nA_S)$ [lb]
ACI 23, 1920 Reinforced Concrete	Steel columns filled with concrete and encased in a shell of concrete Composite columns with cast iron core - Emperger type	f = 18.000 – 70 L/R < 16.000 [lb/in.²] allowable stress on steel section f = 12.000 – 60 L/R < 10.000 [lb/in.²] allowable stress on cast iron section
ACI E-1A-28T, 1928 Reinforced Concrete	Composite columns with cast iron or steel core and Combination columns	allowable stress as in ACI 23 Structural steel columns of any rolled or built up section ... encased in concrete
ACI 501-36-T, 1936 Reinforced Concrete	Composite columns and Combination Columns	with cast iron or steel core $P = 0{,}225\,A_c f_c' + f_s A_s + f_r A_r$ [lb]
ACI 318, 1947 Reinforced Concrete	Composite columns and Combination columns	as in ACI 500-36-T
ACI 318, 1977 Reinforced Concrete	Composite columns	$r = \sqrt{\frac{(E_c I_g/5) + E_s I_t}{(E_c A_g/5) + E_s A_t}}$ Effective radius of gyration

Simplified design method

The proposed simplified design method is based on the simplified method given in Eurocode 4. The design normal force including load factors has to be smaller than the column resistance to normal force:

$$N_{Sd} \leq N_{Rd} \qquad [4]$$

As proposed by Führer (1980) for concrete columns, the resistance of a composite column is calculated as the product of only three terms:

$$N_{Rd} = A \cdot \sigma_{Ri} \cdot k \qquad [5]$$

where

A is the total cross sectional area of the column
σ_{Ri} is the ideal allowed stress, including all safety factors and
k is the buckling factor according to European buckling curves.

The total cross sectional area can be calculated easily, the ideal allowed stress can be tabled exactly considering the column section, the used concrete and the percentage of additional reinforcement, see figure 14. For determination of buckling factor k, the ideal slenderness λ of the column is used instead of the relative slenderness $\bar{\lambda}$.

For steel columns, the relative slenderness $\bar{\lambda}$ can be calculated as the product of the "old" slenderness λ and the reciprocal of λ_a, thus considering the material properties of steel.

$$\bar{\lambda} = \sqrt{\frac{N_{pl,\,Rk}}{N_{cr}}} = \frac{s_k}{i_{id}}\frac{1}{\lambda_a} \qquad [6]$$

where

s_k is the buckling length of the column
i_{id} is the radius of gyration and
λ_a is a factor considering the material properties of steel

$$\lambda_a = \pi\sqrt{\frac{E}{f_{yk}}} \qquad [7]$$

If we apply this principle to composite columns and solve equation (6) to the radius of gyration i_{id}, we can determine an "ideal" radius of gyration of a composite column:

$$i_{id} = \frac{\sqrt{I_a + \frac{I_c}{n_E} + I_s}}{\sqrt{A_a + \frac{A_c}{n_c} + \frac{A_s}{n_s}}} \qquad [8]$$

To avoid by-hand analysis of equation (8), the ideal radius of gyration of a composite column is now calculated as the product of the radius of the steel section and a factor called a:

$$i_{id} = i_a \cdot a \qquad [9]$$

The factor a can be read out of a diagram considering the steel section and the used concrete. Approximately, the influence of additional reinforcement is neglected. For usual column length of 3 to 5 metres (slenderness smaller than 70), the fault of this procedure is limited to 3%. In many cases, results calculated by this simplified method are equal to those calculated by Eurocode 4 method. With the ideal radius of gyration, the ideal slenderness of the column can be calculated:

$$\lambda_{id} = s_k / i_{id} \qquad [10]$$

The complete procedure for concrete filled hollow sections fits on one page **[fig. 5]**, the diagram to determine factor a is inspired by the work of Klöppel and Goder (1957). The European buckling curves are transformed by multiplying the domain values by λ_a, thus reintroducing the material properties of steel.

The limitations of the proposed simplified design method are the same as in Eurocode 4. The most important is the composite condition:

$$0.2 \leq \delta \leq 0.9 \qquad [11]$$

which means that a composite column is a composite column if the steel section carries more than 20% and less than 90% of the column load.

Résumé

An overview of the construction history of composite columns was given, with a special focus on first and temporary application in buildings and a comparative analysis of building regulations. For the proposed approximation procedure, ideas from Emperger, Klöppel and Roik were taken into account, demonstrating the importance of construction and design history for today.

The ultimate load of a composite column is determined as the product of the total area with an effective stress. Buckling safety is provided by using the former slenderness ratio s_k/i_{id} (L/r), where s_k (L) is the length of the column and i_{id} (r) its effective (or ideal) radius of gyration. Furthermore, numerous means are provided to calculate the ideal radius of gyration of a composite column by multiplying the radius of the steel section with a correction factor. The procedure for concrete filled steel pipes fits on one page of paper **[fig. 5]** and can be used both in university education of architecture and by structural engineers for dimensioning and design

of composite columns. In many cases, approximately calculated ultimate column loads are equal to those calculated by Eurocode 4 method. The procedure has been successfully presented to students at RWTH Aachen University (Krauss, Führer, Jürges, 2003, pp. 178-182).

Prospects

Beyond the given example of composite columns, Construction History could play a more important role in the education of both architects and engineers. Former design methods could be studied in all materials, e.g. wood, steel and concrete construction, as they are more graphical to students of architecture. Furthermore, elder methods and procedures to determine the inner forces of statical systems could be studied, to gain the capacity of simple control of computer calculations. A concept for historic and genetic education of the Theory of Structures was presented by Kurrer (2002, p. 455). According to his concept, the presented study represents a historical-logical longitudinal section.

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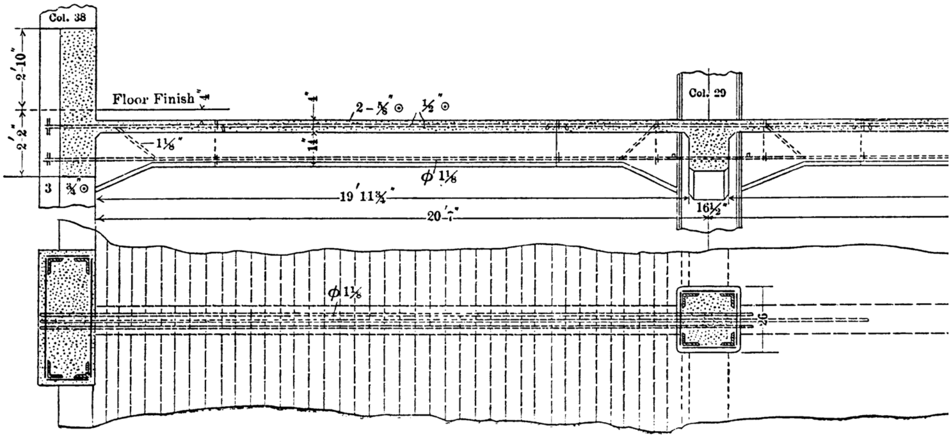


Fig. 1. Mc Graw Building, column and floor section (Burr, 1908, Plate LVI).

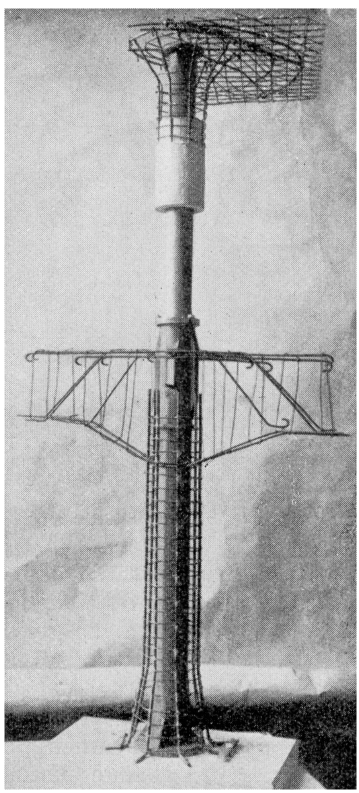


Fig. 2. Emperger column, presented at Leipzig fair in 1912 (Emperger, 1913b, fig. 7).

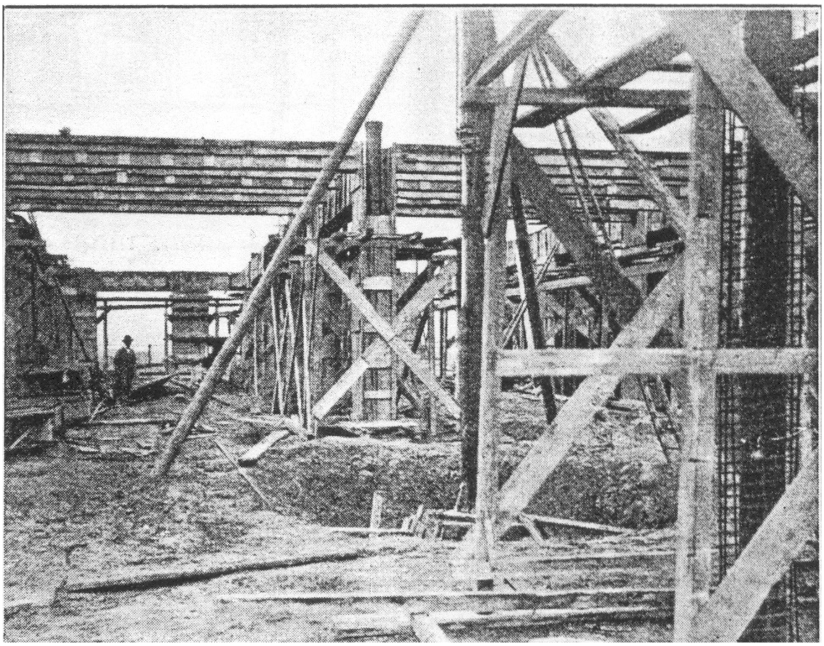


Fig. 3. Ericsson Building, Vienna, 1913 (Emperger, 1913b, fig. 5).

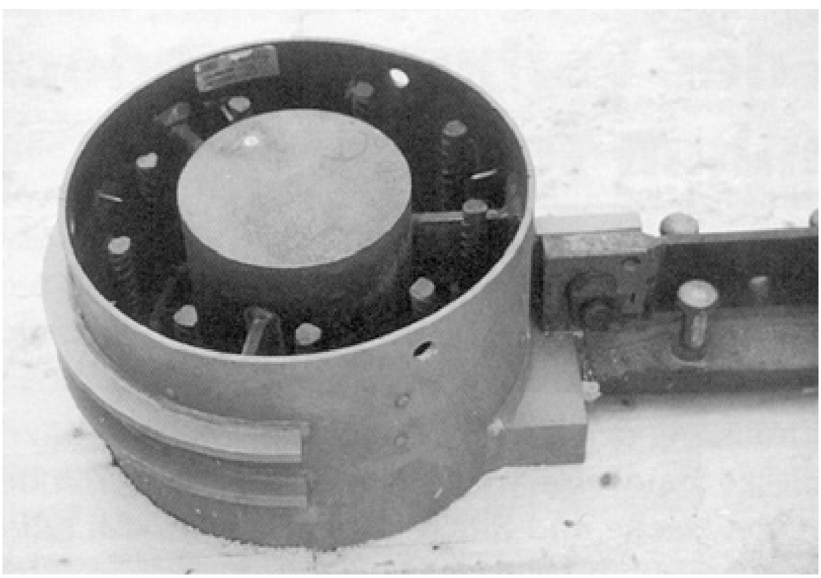
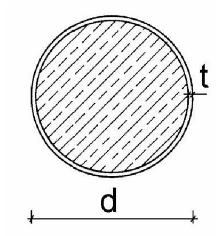


Fig. 4. Millenium Tower Vienna, exterior composite column (Angerer *et. al*., 1999, fig. 3).

Simplified design of concrete filled hollow sections

(Circular and square sections)



$$N_{Rd} = A \cdot \sigma_{Ri} \cdot k$$

A: total area

σ_{Ri}: ideal allowed stress

k: reduction factor

$\lambda_{id} = s_k / i_{id}$ whith $i_{id} = a\ i_a$

λ_{id}: ideal slenderness of composite column

s_k: buckling length

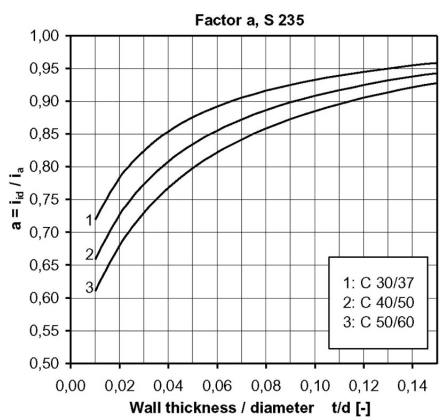
i_{id}: ideal radius of gyration of composite section

i_a: radius of gyration of steel section, if $t/d < 0{,}05$ then: $i_a = 0{,}34d$ (circular) or $i_a = 0{,}38d$ (square)

a: factor

Ideal allowed stress σ_{Ri} [kN/cm²], Concrete filled sections, S 235

t/d	C 30/37 ρ [%]					C 40/50 ρ [%]					C 50/60 ρ [%]				
[%]	0%	1%	2%	3%	4%	0%	1%	2%	3%	4%	0%	1%	2%	3%	4%
0,01	2,77	3,17	3,56	3,96	4,36	3,41	3,80	4,19	4,58	4,97	4,05	4,43	4,82	5,20	5,59
0,02	3,52	3,90	4,28	4,66	5,05	4,13	4,51	4,88	5,26	5,64	4,75	5,12	5,49	5,86	6,23
0,03	4,25	4,62	4,99	5,35	5,72	4,84	5,20	5,56	5,92	6,29	5,43	5,79	6,14	6,50	6,85
0,04	4,97	5,33	5,68	6,03	6,38	5,54	5,88	6,23	6,57	6,92	6,10	6,44	6,78	7,12	7,46
0,05	5,68	6,02	6,35	6,69	7,02	6,22	6,55	6,88	7,21	7,54	6,76	7,08	7,41	7,73	8,06
0,06	6,37	6,69	7,01	7,33	7,65	6,88	7,20	7,52	7,83	8,15	7,40	7,71	8,02	8,33	8,64
0,07	7,04	7,35	7,66	7,96	8,27	7,54	7,84	8,14	8,44	8,74	8,03	8,33	8,62	8,92	9,22
0,08	7,70	7,99	8,29	8,58	8,87	8,17	8,46	8,75	9,03	9,32	8,64	8,92	9,21	9,49	9,77
0,09	8,34	8,62	8,90	9,18	9,46	8,79	9,07	9,34	9,62	9,89	9,24	9,51	9,78	10,0	10,3
0,10	8,97	9,24	9,50	9,77	10,0	9,40	9,66	9,92	10,1	10,4	9,82	10,0	10,3	10,6	10,8



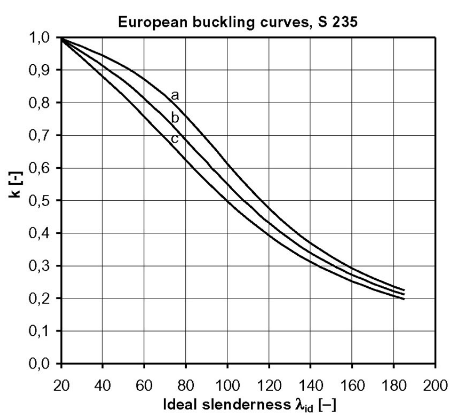


Fig. 5. Simplified design of composite columns (Eggemann, 2003, p. 85).

Hermann Schlimme

Building knowledge encounters emerging natural science: the 'Accademia della Vachia' in Florence, 1661-1662*

The present research has been developed within the *Epistemic History of Architecture* research program, funded by the Max Planck Society, which operated in collaboration with the Max Planck Institute for the History of Science in Berlin, and the Bibliotheca Hertziana, Max Planck Institute for Art History in Rome. The *Epistemic History of Architecture* «is dedicated to investigating the knowledge involved in architectural achievements [...]. It [...] comprises types of knowledge ranging from knowledge implicit in the rules of practitioners to scientific theories and technologies applied to planning and realizing modern architecture. The aim is to reconstruct the systems of knowledge incorporated in the building process itself and their interaction with other knowledge systems» (Bührig *et al*., 2006, p. 7). One of the present writer's principal focal points for the research is the intercourse between emerging modern science and building practice.

In the historiography of Renaissance and Baroque architecture, theory of knowledge always plays a latent role; when an architect's approach to designing is studied, for example, it is usual practice to attempt to determine the treatises he might have known, and the buildings he might have seen. When building typologies, decorative systems, the development of a formal idiom, or for instance, the emergence and evolution of plans, elevations, and perspectival views [*iconografia, ortografia, scaenografia*] in the process of architectural design are investigated, the research essentially retraces the development of the respective fields of knowledge. Only in very exceptional cases does it bring into focus the very notion of *sapere* (knowledge) itself, as for instance, when addressing the training of an architect. But in these cases one normally focuses on the subject's early career, corresponding to the period of the future architect's apprenticeship and training. The project *Epistemic History of Architecture* instead

* This research was published in Schlimme, 2006c. See Schlimme, 2006a for a complete and critical edition of the manuscript, which describes the activity of the Accademia della Vachia. See also Schlimme, 2006b.

considers the architect's entire career as a succession of acquisitions and applications of knowledge. What does it mean to place knowledge itself at the focus of our attention? It means that the history of architecture becomes an investigation of the cognitive abilities of architects and of the persons who were involved in construction sites. This also includes places where the acquisition, exchange, and application of knowledge occurred.
The construction site has already been seen for some time in the scholarship not only as an important phase in the genesis of a specific building, but also as an object worthy of study in itself. In epistemic history, construction sites become "places of knowledge"; other such sites include workshops, architectural studios, academies, discussion circles, and so forth. The libraries and private collections of the architects and the entire culture of debate that existed in an historical situation is understood as a field of reflection for knowledge. It is therefore necessary to explore the exchanges of knowledge between the construction industry and other fields of knowledge such as banking or mechanics. Seen in this light the history of architecture explores the historical dimension of problem-solving strategies, as well as the instruments, equipment, tools, and methods used by architects and other building professionals. The points of contact between building practice and natural philosophy, especially mechanical science, in recent times have stimulated new interest among Italian and foreign scholars[1], and this issue is especially important to the Associazione Edoardo Benvenuto.
One goal of all these approaches is to bridge the divide that exists be-

[1] Over the past twenty years, various focuses of activity can be mentioned, such as the book series *Between Mechanics and Architecture*, Basel 1995- (initiated by Patricia Radelet de Grave and Edoardo Benvenuto), and *Studies in the History of Civil Engineering,* Aldershot 1997-2001 (Joyce Brown general editor). Reference should also be made to the activities and publications of the Associazione Edoardo Benvenuto, the Archive of the Art and Science of Construction (Genoa), the Museo Galileo (Florence), the International Seminar *Teoria e pratica del costruire: saperi, strumenti, modelli* (Ravenna 2005), the congresses of the different national societies devoted to Construction History and the International Congresses on Construction History (Madrid 2003, Cambridge 2006, Cottbus 2009, Paris 2012, Chicago 2015, Brussels 2018, Lisbon (*online*) 2021) with their respective, published proceedings. Becchi, Carvais and Sakarovitch, 2018 give an overview over the research field. For further bibliography see Schlimme, Holste and Niebaum, 2014, i.e., the chapter "Innovations-Anstöße: Interaktion zwischen Bauwesen und entstehender moderner Naturwissenschaft", pp. 341-346.

tween two areas: on one hand, mechanical science, and in particular its history, traditionally treated as part of the history of science, and on the other, building technology, and in particular its history, which is generally considered as part of the history of architecture. It is true that even in the seventeenth century these worlds were separate, each with its own specific institutions, but as we shall see there was a great deal of contact and exchange between the exponents of the institutions; indeed, it is this exchange that today in the equally separate historical disciplines tends not to be seen, and hence not to be studied. Under these circumstances, collaboration between the history of science and the history of architecture, as put into practice in our program, may be extremely useful, but it is above all the epistemic approach, that is, the approach of the *history of knowledge* in more general terms, that can make a contribution here. We approach the history of architecture as a history of the discussion between fields of knowledge, as in a history of interactions of knowledge. In what follows, I would like to offer an example of the kind of results that can emerge through the use of this approach.

What were Florentine institutions like in the 17th century? Technology and the construction of buildings were not academic disciplines. We are informed about the organizations that were inspired by the emerging field of natural science, such as the Accademia del Cimento in Florence (1657-1667)[2], where experiments were carried out in an attempt to understand the laws of nature. Construction technology was institutionalized in craft guilds as well as by the Parte Guelfa and the Ufficiali di Torre (Cerchiai, Quiriconi, 1976; Pansini, 1989; Schlimme, Holste, Niebaum, 2014, pp. 121-126), the most important building authority in grand-ducal Tuscany. Architects, painters, and sculptors had broken away from trade organizations. In Florence, by the second half of the sixteenth century they were organized in the Accademia del Disegno[3]. Parallel to the creation and development

[2] See Galluzzi, 2001 and Boschiero, 2007 for information on the Accademia del Cimento and on science in the Grand Duchy of Tuscany in the second half of the 17th century. See also Schlimme, Holste, Niebaum, 2014, pp. 341-346.

[3] The Accademia del Disegno has been studied intensively, among others in the monographs of Ward, 1972; Jack, 1976; Barzman, 1985; Barzman, 2000 and Waźbiński, 1987 and in further studies of Barzman and in the papers of Bencivenni, 2001; Burioni, 2004; Carrara, 2008; Schlimme, 2009a; Schlimme, Holste, Niebaum, 2014, pp. 139-143 and Meijer, Zangheri, 2015.

of associations and professional institutions, interaction was common between specific fields of expertise, and this exchange was certainly of no less significance. Encounters between the disciplines, however, often took place at a remove from the institutional context and in many cases in private, as a result of which less documentation is often available to us[4].
A circumstance of this kind is represented by the Accademia della Vachia, that existed from 1661 to 1662, and which is almost completely forgotten today. Apart from a small number of earlier citations[5], it was treated for the first time by the present writer in 2006 (Schlimme, 2006a; 2006b; 2006c) and is today acknowledged in historiography[6]. The starting point is an unpublished manuscript that the present writer located in the Biblioteca Nazionale in Florence. The manuscript bears the title *La Risoluzione di più Problemi stati proposti nel Accademia del Sig. Abate Ottavio della Vachia L'anno 1662 con i Nomi di chi propose et di chi ha Risoluto*[7].

[4] For examples of this, cf. Galuzzi, 1977; Becchi, 2002, p. 30, who – based on the example of later 17th-century France – studies the relationship between architecture and applied science; Schlimme 2006d; for the shipbuilding context: Renn, Valleriani, 2001.

[5] In his *Inventari dei manoscritti delle biblioteche d'Italia*, Mazzatinti refers to the Accademia della Vachia eand mentions the problem of the chronometer (problem 22; Mazzatinti, 1892-1906, vol. VII, 1897, p. 215). In 1919, Giovanni Giovannozzi mentions the manuscript in an article on Cosimo Noferi, but gives no further information regarding the academy (Giovannozzi, 1919, pp. 117-119). Giovannozzi presents very brief descriptions of three problems, and promises an essay, which never appeared. In 1960, Michelangelo Muraro mentions the manuscript in an essay on the painter Noferi (Muraro, 1960, pp. 86-89). In a series of cases there are references to the academy without it being mentioned by name. In his book on the engineer Bartolomeo Vanni (1662-1732), Luigi Zangheri cites one of the academicians, Giuliano Ciaccheri, in a note, quoting the location of the manuscript (Zangheri, 1977, p. 28, footnote 80). In his article on Ciaccheri in the *Dizionario Biografico degli Italiani,* Zangheri states that Ciaccheri was educated "alla scuola degli epigoni di Galileo Galilei" (Zangheri, 1981). In an essay on Ciaccheri in the *Allgemeines Künstlerlexikon* (AKL), Partsch quotes Zangheri (Original text: «Lernte in der von den Schülern Galileis geführten Schule Archit. und Ing.-Wesen», (Partsch, 1998), and in the article on Jacopo Foggini in the same AKL, Partsch mentions the academy of the Abbate della Vecchia (Original text: «[...] war der Accad. Geometrica des Abtes Della Vecchia [sic] verbunden, in der die Mitgl. geometr., archit. und mathemat. Probleme erörterten» (Partsch, 2004).

[6] See e.g. *Enzyklopädie der Neuzeit,* Meyer, 2016, pp. 311-312.

[7] Biblioteca Nazionale Centrale di Firenze (=BNCF), Ms., Fondo Nazionale (ex Nelli), II_46; english translation of Ms. title: *The resolution of many problems that were proposed in the academy of Sig. Abbate [Mr. Abbot] Ottavio della Vacchia, [in] the year 1662, with the*

Wherever reference is made to the academy, “Vachia” is spelled with a single “c”, as it appears in the manuscript, while the family name “Vacchia” found in the sources always presents the double “c”. The manuscript is from the collection of Giovanni Battista Clemente Nelli (1725-1793) and comprises 159 pages, including 51 full-page images, many with a colour wash. The volume measures 33 x 23 cm, and its pages, partially folded, are of irregular format and different qualities of paper. The bound manuscript was organized to compile the records of the academy.
We know the names of 13 members of the academy, and in most cases also the individual with whom they correspond. The first on the list is the clergyman Ottavio della Vachia, member of a Florentine patrician family, in whose house the meetings probably took place. The key members were the architect and mathematician Noferi Cosimo, a follower of Galileo; the clergyman and mathematician Domenico Fontani; Francesco Barzini, a professor of astronomy at the “studio di Firenze”, that is, the university of Florence; Giovanni Andrea Albizzini, who was a professor of philosophy at the university of Pisa and an opponent of Aristotelian philosophy; Jacopo Ramponi, an engineer of the Parte Guelfa specializing in military architecture; the youthful Giuliano Ciaccheri, then still a student and who would also later become an engineer for the Parte Guelfa; and Jacopo Maria Foggini a sculptor and Accademico del Disegno in Florence and the uncle of the better known Giovanni Battista Foggini[8]. The historical significance of the Accademia della Vachia, however, lies less in the importance of its individual members, than in its being a forum where persons from different professional and institutional backgrounds interacted. From the watermarks on the paper it has also been possible to establish a link to the Medici family (Schlimme, 2006c, p. 61 and footnote 5). As a rule, the academicians met on Sundays. Each Sunday, they would set each other problems for resolution by the following week.
We have 49 of these problems and their respective solutions in the manuscript. The most frequent issues are represented by mathematical problems, often conic sections and also perspective geometry, the construction of machines and measuring instruments. Also treated are technical

names of who proposed them, and who resolved them.

[8] See Schlimme, 2006c, pp. 64-67 and footnote 6 in the present text for further information on the members of the academy.

issues related to large-scale sculptures. Many of the problems are also relevant to the construction industry: roofing, structural conversions of extant buildings, fortifications, hydraulic constructions, fountains, theaters, and bridges[9].

It is the presence of Cosimo Noferi, above all, that enables us to locate the academy in the cultural context of the epoch. Noferi was a Florentine mathematician, architect and artist. He may be seen as one of the first architects to have adopted the approach of the emerging modern sciences, attempting to apply it to the construction industry. Noferi was intimately acquainted with the work of Galileo and Torricelli, although he was not the direct disciple of either, given that he was born around 1635. In the foreword to the first part of the *Travagliata Architettura,* a manuscript in four volumes published by Antonio Pellicanò (Pellicanò, 2005), he frequently incorporates discussions already addressed within the Academy of Vachia, and affirms: «I will not just lay down the rules for the things I aim to do; I will lecture on them and prove their validity through their underlying geometric and natural logic. These discourses, therefore, are not devised by some simple practician, but by a speculative philosopher; and I will prove that the architect has to be a person trained in this way»[10]. In other words, Noferi announces that he will describe not only techniques and phenomena, but also explain their underlying geometric and natural causes or reasons.

Noferi explains his approach by bluntly distinguishing it from the Aristotelian tradition. The architect «should consider the forces, movements, weights, proportions, time, and what the intellect suggests to him in order to examine a certain phenomenon [*accidente*]. This is the path he should follow to reach such perfection that, though others may indulge in mere idle talk and have more eloquence and memory, he [the architect] will be able to silence any peripatetic [Aristotelian] arrogance with wisdom and circumspection, and with a few, brief words»[11]. Noferi relies on episodes

[9] See Schlimme, 2006a for a critical edition of the entire manuscript.

[10] Italian Original: «Non solo di quello che propongo di fare do la sola regola, ma sopra vi faccio lezione, et discorso, e provo con ragioni geometriche, o ragioni naturali, e così tali discorsi non sono di semplice pratico ma da speculativo filosofo, che tale, come proverò deve essere il perito architetto»: BNCF, Ms., Galileiani 122, Proemio, f. 2*r*; see also Discorso quarto, f. 62*v*.

[11] BNCF, Ms., Galileiani 122, Discorso primo, parte seconda, f. 15*r*. Italian Original: «[…]

such as Galileo's discussions with the workers of the arsenal in Venice. Galileo proceeded from the experience of the workers, who claimed for instance that if the scale of a structure (in this case, a ship) was increased, it was not sufficient merely to enlarge all its elements proportionally. They had to be enlarged to a greater degree in cross-section (Renn, Valleriani, 2001). This was a point of departure for Galilei's materials science, as described in his *Due nuove scienze* of 1638 (Galilei, 1638).

That Noferi, like the aforementioned academician Albizzini, advocates Galileo and clearly rejects Aristotle, is attributable at least in part to the effect of Galileo's own biography. The latter's break with the Greek philosopher and scientist, however, was not as pronounced as it then appeared, or as it appears from the immediately subsequent historiography. For example Bernardino Baldi, the Aristotle commentator, had anticipated many of Galileo's ideas, as has recently been clarified by Antonio Becchi (Becchi, 2004). Galileo moreover, does not mark the beginning of the development but rather stands at the end of a long line of development that had begun at the latest in the mid 15th century with Nicolaus Cusanus (Nicholas of Cusa) and his *Idiota: De Experimentis Staticis*, in which Cusanus shows a deep interest in the knowledge of artisans and their outlook on life (Cusanus, 1937; De Boer, 2003). Among the many others who followed this approach in the 16th century, were for instance Giulio Agricola and Bernard Palissy. The methods and techniques of artisans were seen as a pool of extant experimentation with nature - and experimentation was the method of approach of natural science in the modern sense, whereby to gain knowledge of the laws of nature.

With his well-known *Novum Organum*, published as part of the *Instauratio magna* in 1620[12], Francis Bacon initiated an experimental approach to the understanding of nature. In the now much less known *Catalogus Histori-*

considera le forze, i moti, i pesi, le proportioni, i tempi, e quello che gli suggerirà l'intelletto da specolare sopra di tale accidente, e questa sarà quella strada, che l'incamminerà ad una tal perfetione, che potrà ben altri haver maggior chiacchiera, et eloquenza, o memoria, ma egli saggio, e prudente con brevi e poche parole saprà quietare ogni peripatetica alterezza [...]»: BNCF, Ms., Galileiani 122, Discorso primo, parte seconda, f. 15*r*.

[12] Bacon, 1620; Bacon, 1992. The *Instauratio Magna* contains the *Novum Organum*, which is intended as *secunda pars instaurationis*. The full title is *Novum Organum sive indicia vera de interpretazione naturae*.

arum Particularium, published as an appendix to the *Novum Organum*[13], Bacon called on natural philosophers to systematically and comprehensively document and study the knowledge accumulated in the mechanical arts and crafts, which he saw as a pool of ready-made experiments in the realm of nature. Bacon even provided a documentation programme for no less than 130 fields of knowledge, with almost a dozen building crafts, including the *historia* (description) of quarrying, carpentry, wrought-iron work and the production of glass, bricks, mortar and cement. According to Bacon, the laws of nature were also easier met with in the arts, where man grapples directly with nature, than in nature itself in its original state. If for a certain period natural scientists investigated the crafts and techniques with particular interest, from the middle of the 17th century onward, we find an interaction between science and building industry: an interest developed for applying the methods of emerging natural science to construction, as may be seen, for instance in the backgrounds to the foundation of the academies of London (1660) and Paris (1666), and in Italy, especially in the Accademia della Vachia. This process resulted in a series of innovations.
Fields of knowledge that had not been deemed worthy of presentation in architectural treatises but which were crucial for the daily operation of construction sites now became the focus of interest, as for example the complex knowledge involved in the restoration and conversion of existing buildings by means of extensive operations to their static structure. The problem 12 of the Accademia della Vachia concerns two parallel spaces with barrel vaulted ceilings, which were to be transformed into a single, large barrel vault without demolishing or damaging the floors above, which were divided into a series of smaller spaces. Solutions propose a complex shoring structure for the storeys above which would have allowed the larger vault to be rebuilt from scratch (Noferi). Fontani instead proposes to proceed *braccio* by *braccio* with a sort of moving centering[14].
Another example is the quantification of the experiential knowledge of craftsmen. A problem discussed in the Accademia della Vachia concerned

[13] In the *editio princeps* of 1620 (Bacon, 1620; Bacon, 1992), the *Novum Organum* is followed by a *Descriptio historiae naturalis, et experimentalis*, a text titled *Aphorismi de conficienda historia prima* and a *Catalogus historiarum particularium*.

[14] *In extenso* see Schlimme, 2006a, pp. 192-194.

the coordination of the supply and outlet of a fountain that was needed to maintain the basin well filled but never overflowing. The academicians did not want to simply rely on the use of an overflow, and so consulted recent treatises on hydromechanics to calculate the inlet and outlet of the water and to predefine the tube diameters. The new knowledge created within the emerging natural sciences was applied to technical problems, which was definitely a step towards a science-based form of construction[15].

For the roof of the church of San Giovannino in Florence, the academicians developed a new type of *capriata*. The Jesuits' church[16] and adjacent convent had been built starting in 1579 by Bartolomeo Ammannati. The height limit, determined on the basis of the dimensions of the Palazzo Medici Riccardi opposite the church, had imposed the construction of a flat ceiling within the church. Through the mediation of Leopoldo de' Medici, in 1655 the Jesuits received the Grand Duke's permission to raise the roof of the church by six braccia (about 3 meters 50) and to insert a lightweight wooden barrel-vault ceiling[17] **[fig. 1]**. Between 1662 and 1665 the roof was rebuilt[18]. To construct the loftiest and impressive barrel-vault ceiling possible, the vault had to extend upward into the space of the roof.

[15] *In extenso* see Schlimme, 2006a, pp. 224-228.

[16] In 1773, the Jesuits were expelled from Florence, and the church and convent were given to the Piarists (*Scolopi*). For more information about the church, see: Bösel, 1985; Boccia, 1989; Bencivenni, 1996 (each with bibliography).

[17] As early as 17 April 1655, the General of the Jesuit Order sent his compliments in a letter to the Grand Duke: Bösel, 1985, p. 81 (Archivum Romanum Societatis Iesu = ARSI, Roma, 31 II, f. 421). Bösel says that the Grand Duke allowed an addition of six *braccia* in height. This is never stated explicitly in the sources, but the new roof truss does effectively have roughly this height. Bencivenni, 1996 briefly mentions the raising of the roof, while Boccia, 1989 cites Bösel, 1985 and concentrates on the original Ammannati designs.

[18] From 1656 to 1659, Luigi Lamberti was the Superior in Florence. He planned to erect a dome over the tribune of the church (Bösel, 1985). On 15 June 1658, the General of the Jesuits ordered Lamberti to halt these plans (ARSI, Roma, 32 I, f. 92). After this, Lamberti did not dare to undertake any building activity. On 4 January 1660, his successor, Lidano Colonelli is first mentioned (ARSI, Roma, 32, I). Unfortunately, we have no further written evidence for the building of the roof. There are no sources in the Archivio di Stato di Firenze (=ASF). In the ASF "Compagnie religiose soppresse da Pietro Leopoldo", Patrimonio Ex Gesuitico, pezzi 1064 and 1065, there are only records of the construction of the convent in the 17th century. The church was reconsecrated on 28 October 1665; Bösel, 1985, p. 81.

The Accademia della Vacchia dealt with this problem. The connection with San Giovannino becomes clear from a statement by Noferi in a part of his *Travagliata*, which he evidently wrote after the Accademia della Vachia experience[19], and from the correspondences in dates and circumstances. The task of the Academy is recorded as follows in the manuscript: «Given the form ABD of a vault to be constructed, and the inclination of the roof FE distant in CD [by] a *braccio*, the roof truss has to be conceived in such a way that it will not interfere with the construction of the above-mentioned vault»[20] **[fig. 2]**. The roof impost had to lie considerably below the apex of the vault in such a way that it would not possible to use the normal *capriata* with a tie beam[21] **[fig. 3]**.
Taccola and Francesco di Giorgio Martini were among the first to have conceived a *capriata* without tie beam, which is stabilized through vertical post and struts. In his *Travagliata,* Noferi proposes a *capriata* with two posts and without tie beam, which would seem to reflect the same idea (**fig. 4**, right column, second from top)[22]. Many of the academicians' proposals were based on the so-called "*capra*", a term used in the first half of the seventeenth century to describe a *capriata* in which the tie beam is attached not to the foot of the rafters but instead at their halfway point, creating a figure similar to an 'A'. In his *Exercitationes* of 1621, Baldi ad-

[19] Although the Accademia della Vachia is not mentioned in the source material relating to the new roof construction for San Giovannino and conversely the church is not mentioned by name in the manuscript describing the activities of the academy, the two are certainly connected. The decisive link in this respect is Cosimo Noferi's *Travagliata Architettura*, BNCF, Ms., Galileiani 122, Discorso quarto, where the author speaks of the roof trusses that "one wishes to use in San Giovannino", providing illustrations of them as well, and also Fontani's solution which was then adopted (fig. 4 top left). Since Noferi mentions the trusses intended for San Giovannino in Florence, the date of this part of the *Travagliata* cannot be earlier than 1662, although the fourth part of the *Travagliata* can be dated with certainty to 1658. This is not necessarily a contradiction, however, since one cannot be sure that Noferi wrote his treatises in the order in which he subsequently arranged them in the *Travagliata*.

[20] BNCF, Ms., Fondo Nazionale (ex Nelli), II.46, f. 45*v*. Italian Original: «Data la centinatura ADB d'una volta da farsi, et il pendio del tetto FE distante in CD un b(racci)o si deve fare il cavalletto in modo che non impedisca la fabbricazione della su(dett)a volta», BNCF, Ms., Fondo Nazionale (ex Nelli), II.46, f. 45*v*.

[21] For roofs in Early Modern Italy see Valeriani, 2006. For more details on this case see Schlimme 2006c.

[22] Cf. Mussini, 1994, pp. 382-385; BNCF, Ms., Galileiani 122, Discorso quarto, f. 65r.

vises against using this type of construction because as he points out, the free bases of the rafters function as levers, imposing additional forces on the joints between the elevated tie beam and the rafters[23]. The academician Albizzini had a copy of Baldi's work in his private library[24]. Serlio had designed roof structures similar to the *capra* in his architectural treatise, although the roofs are much steeper, and he declares these structures to be typical of France. Unlike Baldi, Serlio did not study the structural aspects of this constructive solution (Serlio, 1575, p. 198). From the illustration, however, it seems that the elevated tie beam had to be connected to the rafters by means of *braces*. From a structural point of view this is efficacious, because it reduces the length of the levers formed by the unattached bases of the rafters.

After summarizing the problem, Noferi proceeds, describing his three solutions of which the first is more or less intended to illustrate the problem yet again[25] **[fig. 5]**. The second solution is based on the idea of the *capra*. He writes: «The problem is the thrust which they exert at their flanks, especially the first roof truss. The second one does not exert so much thrust because of the collar beam LM, which we can, however, substitute in two ways. The first one is with iron stakes N, O (**fig. 5**, "1°."), which fix the rafters to the supports and the bedding, and also penetrate part of the masonry. These stakes should have a form similar to the *alaguto* [word unclear] or to the nail. The second way is without iron stakes, but with stakes PQ made of oak or chestnut (**fig. 5**, "2°."). These have a half-heading R, through which they are hooked in, in a way that fixes together support, bedding and a part of the wall[26]». The weight of the crown of the wall to which the rafters are connected, prevents the rafters themselves

[23] Discussed in Becchi, 2004, p. 84 et seq.; *ivi*, glossary, 129.

[24] ASF, Archivio Mediceo del Principato, pezzo 5545, ff. 10-15.

[25] BNCF, Ms., Fondo Nazionale, II.46, f. 46*r*. For more details, see Schlimme, 2006c.

[26] BNCF, Ms., Fondo Nazionale, II.46, f. 46*r*, 46*v*. Italian Original: «La difficoltà è dello spingere che fanno ai fianchi, particolarmente il primo Cavalletto, il che non fa tanto il secondo mediante lo traversone LM, che però potremo supplire in due modi il primo è con li pali di ferro N, O che inchiodino insieme i puntoni con li mensoloni, et il letto, trapassando anco in parte della muraglia quali pali sieno di figura simile alaguto, o chiodo. Il secondo modo senza il palo di ferro e che eletto di quercia, o di Castagno lo perno PQ al quale fattogli il mezzo capo R q(ues)to [46*v*] li impernerà in modo che leghi il mensolone, il letto, et parte della muraglia».

from moving. In his solutions to other problems proposed to the academy, Noferi had likewise suggested the use of the mass of the masonry itself as a counterweight.
The solution preferred by Noferi (**fig. 5**, “3°.”) consisted in building a steeper roof, which would have permitted a normal *capriata* to be used, as he explicitly states, with elongated rafters[27]. In this case Noferi utilizes a combined strategy: blending solutions that were already known and adapting them to new situations, in this case the normal *capriata* and the *capra*. Since Noferi reinforces the lower portions of the rafters, it seems probable that he recognized that these elements were subject to deformation. His affirmation was immediately called into question, however, since this was already customary in the case of *palladiane*, though for completely different static requirements.
Foggini’s solution seems indebted to Serlio’s illustrations. This would explain why Foggini’s proposal foresaw a steeper roof than what was usual for Florence (Schlimme, 2006c, fig. 15, p. 78). Foggini had understood the play of forces in the system he was proposing, but did not have a clear idea for the design of the joints between the elements: the dovetail joint with which the elevated tie beam is connected to the rafters only functions if the doubled rafters are fixed together with iron buttstraps. The latter elements seem in fact to be represented in the design by Foggini. The anonymous solution combines a series of countermeasures (Schlimme, 2006c, fig. 16, p. 79). The two rafters seem designed on the model of two cranes that protrude independently of each other and which are joined at the top. The rafters are supported by beams that are shorter and slightly more inclined. The rafter structure is connected to the wall by a series of iron straps and vertical wooden members. With this solution a counterweight would be created that was analogous to Noferi’s solution. It seems that Noferi adopted this anonymous solution when he later proposed still another solution **[fig. 6]**.
In January 1662 the same problem regarding the roof was again proposed, though with an important modification. In the first version, the problem required the construction of an *ex novo* roof with new *capriate*; in the revised version, it was required that the pre-extant roof structure continued to be used. As in the first version of the problem, here, too, it was

[27] BNCF, Ms., Fondo Nazionale, II.46, f. 46*v*.

necessary to insert a light wooden vault into the space created by the roof, so that in the new version of the problem it became necessary to cut the tie beam and the vertical post of the extant *capriate* and introduce new members to stabilize the remaining structure *in situ*[28].

Noferi explains his solution as follows: Since the struts are no longer available to support the rafters in the center, other beams are introduced to support them. These new beams join the mid-point of the rafters with the wall below the impost of the roof. In the first solution, to the left, the last part of the tie beam is still present **[fig. 6]**. In the second solution, to the right, the old tie beam is more radically truncated and does not interfere with the new beam[29]. Unlike the strut of a normal *capriata*, this new element is tilted in the opposite direction, at the same time reducing the horizontal thrust of the rafters. In essence, this idea works and the solution seems feasible. Noferi rightly states, however, that the new beams that replace the struts apply a powerful horizontal thrust onto the walls. Noferi probably believed that the weight of the entire upper part of the wall between the impost of the original roof and the site where the new beam was inserted could function as a counterweight. In addition, he calculated that this counterweight could resist against the outward thrust of the whole roof of the complex. It is only possible here to allude to the solutions of Filippo Morosi and Giuseppe Balatri for this second version of the problem (Schlimme, 2006c, pp. 83-85).

While all the solutions considered up to now attempt to conserve an already greatly compromised roof structure by introducing secondary members, Domenico Fontani proposed to transform the remnants into a new *capriata,* based coherently and efficaciously on a different structural principle. Although what was being treated was a work of conversion, it seems not to be such. Fontani's proposal is set forth in three alternative versions **[fig. 7]**. The first, drawn at the bottom of the sheet, shows the two main rafters, which determine the roof line. These are supported by a second pair of rafters in the form of two parallel boards. This second pair of rafters is attached to the foot of the main rafters. From here they rise at an

[28] Non abbiamo una descrizione del problema, ma questa idea principale emerge dai commenti di Giuseppe Balatri, BNCF, Ms., Fondo Nazionale (ex Nelli), II.46, f. 122*r*, e Cosimo Noferi, *ivi,* f. 118*r*.

[29] *Ivi*, f. 118*r*.

angle that is slightly less than that of the main rafters, and intersect below the ridge of the roof where they are fixed to what remains of the vertical post. From there, the secondary rafters continue till they intersect with the main rafters on the opposite side of the roof. In all three of the intersection points, the elements are connected to each other by means of a large number of nails, thereby creating torsionally rigid joints that can withstand rotation. The load-bearing principle is that the horizontal thrust is absorbed within the *capriata*, with the ridge detail designed as a rigid angle at the top of the roof: the two main rafters are subject to compression, the lower pair to tension stresses. Additionally, all of the elements are subject, at their various points of intersection, to bending moments.

In Fontani's other two solutions, the lower pair of rafters is completely replaced with long iron straps, while in the third case the rafters are still wooden, but connected to the original rafters with iron elements. The iron straps can only withstand tension loading, and are not fixed to the original rafters in a torsionally rigid manner. In this way, the load-bearing principle is reduced to compression forces on the original rafters and tension forces on the iron straps. The original rafters are still subject to bending stresses, but the non-rigid, rotating joints mean that not all the members are subjected to these stresses. As a result, the *capriate* are thus somewhat less rigid than those made entirely of timber. Despite the functional quality of this solution, Fontani did not introduce elements that would replace the missing struts, which in the original *capriata* supported the main rafters in the middle. On the contrary, the main rafters are subject to additional loading: the pairs of secondary rafters attached to the main rafters increase the bending moments of the latter. It is doubtful whether the main rafters of the original *capriata* could have withstood this added load. To what extent the academicians were also aware of this aspect is unknown.

Fontani's concept was adopted **[fig. 8]** for the roof, though significantly not as he had intended, that is, as a solution for altering the existing structure, but rather as a basic concept for the complete rebuilding of the roof. It seems unlikely to be coincidental, however, that the rafters as they were carried out in the church roof are of exceptionally large size. Whether this was intended as a countermeasure to limit the general risk incurred by what was for the time an unfamiliar structure, or as a specific response to the considerable bending moments, remains an open question. The *capriate* as currently installed in San Giovannino have not gone completely unremarked - they were, for instance, part of a building survey carried out by Boccia in 1998 -, but up to now they have never been described or

analyzed. Each single *capriata* has a span of about 12.80 metres, and is constructed from four wooden beams that measure roughly 27 cm wide and 37 cm in height. Two of these beams are the actual rafters. Springing from the top of the walls where they are seated on timber consoles and meet on the roof ridge. The roof has a slope of 18/19 degrees, which corresponds to a 1/3 gradient, as recommended by Noferi in his *Travagliata*[30]. As in Domenico Fontani's solution, the other two beams, which are slightly longer, rest on the same timber consoles in the wall, but are not so steeply inclined. They intersect below the ridge line of the roof and continue on to connect with the main rafters on the other side of the roof. These four intersections are in the form of halved joints, marking an improvement over the nailed solution proposed by Fontani. Unfortunately, in 1993-1994 whoever restored the roof decided to introduce an unnecessary steel skeleton structure to support it. It is to be hoped that this will not damage the roof timbers, which over more than three centuries have shown no visible deflection.
Apart from the unique quality of this *capriata* in Italy, a number paradigm shifts can also be observed. Instead of using traditional carpentry joints, the academicians used halved joints [*gionzione a mezzo legno*]. C*apriate* were normally transported onto the roof piece by piece and assembled *in situ*. Marks were therefore required to indicate the positions of the joints. No such marks are found on the roof elements in San Giovannino. It must also be said, however, that altogether such marks were not much used in Italy, since Italian *capriate* are less complex than roof constructions of northern Europe (Valeriani, 2006). There is evidence, however, that the *capriate* were put together on the ground and then hoisted as units onto the roof[31]. Indeed, among the problems studied by the Accademia della Vachia is a description illustrated by Cosimo Noferi elucidating the "transport of pre-assembled capriate", in this case one of classical construction, onto a rooftop using a pole crane. The drawing **[fig. 3]** even shows a narrow street that could actually represent the Via de' Gori that runs along the north side of the church of San Giovannino[32].

[30] BNCF, Ms., Galileiani 122, Discorso quarto.

[31] Thanks are due to Frank Becker for drawing my attention to this question.

[32] BNCF, Ms., Fondo Nazionale (ex Nelli), II.46, f. 59*r*. The problem is posed and solved by Cosimo Noferi. Schlimme, 2006c, p. 86; Schlimme, 2006a, pp. 207-208.

The *Travagliata Architettura* contains a kind of report of the discussions regarding the roof of San Giovannino, in which each member was invited to discuss the approach to the problem and the solutions. While the public discussion of building issues is nothing new, we can nevertheless see a paradigm shift. New knowledge was being created in an orderly fashion, and the plan was to make them available for everybody's use through publication. Noferi, however, is not entirely sincere. While showing Domenico Fonatani's solution (**fig. 4**, top left) he fails to mention his name and to illustrate all the solutions, instead inventing others of his own. Noferi never published his own book, however, and thus it is not surprising that this type of construction was not carried out anywhere else in Italy, at least as far as the present writer knows, during the period immediately following the rebuilding of the roof of San Giovannino. It can be compared, however, with similar constructions (both earlier and later) in England and Germany[33].

The case that has been presented here belongs to the core subjects in the history of technology in the classical sense, and represents an additional chapter on the *capriata* without tie beam. This is a history that still had a long evolutionary road ahead of it, up to the Polonceau trusses and beyond. As the present writer has attempted to show, utilisation of the specific approach of epistemic history has at the same time contributed a number of new elements. The approach has enabled the discovery of archival sources that might otherwise have fallen into oblivion between the histories of science and architecture, it has shed light on the knowledge and the way of thinking of the people involved in the discussion, and has demonstrated how in a specific case the juxtaposing of knowledge generated new knowledge. It has also shown where the academicians had to abandon the accustomed way of doing things. Besides coming up with a new form of roof construction, the Della Vachia academicians did not fall back on artisanal joints, and furthermore seem to have hoisted the whole pre-assembled *capriate* up to the roof, apparently in a form of building site rationalization.
The aspect of special interest within this episode is that it enables us to trace the process by which a technical solution was developed and adopt-

[33] Yeomans, 1992; Ostendorf, 1908; Holzer and Köck, 2008; a more detailed analysis is found in Schlimme, 2006c, pp. 87-88.

ed in the debate of the period. We have seen an environment of open and public debate animated by the desire to apply the approaches of the emerging modern natural sciences and involving all the experience and knowledge of all who were present, experts or not, in investigating the reasons that roof constructions behave as they do. Undeniably, the result also comprises the uncertainties that are inevitably part of any history, and that we have also been able to observe in the *modus facendi* of the academicians.

After the work on the Accademia della Vachia, the present writer has continued investigating the interaction between emerging modern science and the building sector, especially that between the application of modern science to architectural enterprises, on the one hand, and the practical knowledge and experience of architects and craftsmen on the other, focusing in particular on dome construction (Schlimme, 2006d; Schlimme, 2009; Schlimme, 2011; Döring-Williams, Schlimme, 2011) and the emerging field of materials science. The growing availability of standardised values for characterizing building materials during the 17th century and especially from the 18th century onward, shifted knowledge about the nature of materials from the realm of the local craftsman's experience to the world of planners and engineers, who use methods of calculation deriving from the application of the approach of modern natural science to building (Schlimme, 2008; Schlimme, Holste, Niebaum, 2014, pp. 262-267).

The author applied the approach developed within the project Epistemic History of Architecture and during the studies concerning the Accademia della Vachia to further topics concerning the interaction of different knowledge inventories. Leonardo da Vinci repeatedly dealt with centerings for vault construction in his manuscripts. He was certainly inspired by the construction of St Peter's, which he was able to follow during his stay in Rome (1513-1516). Leonardo linked the knowledge of craftsmanship of his time with other fields of knowledge such as geometry and natural philosophy. He was, in fact, not only interested in the improvement of centering construction, but centerings served him as cognitive instruments for studying the structural mechanics of the arch and as material for his research for geometric systematics and seriality. Following on from such considerations, Leonardo then proposed alternative designs for centerings that are at the same time firmly anchored in Renaissance building site practice and timber construction logic (Schlimme, 2021).

The approach of epistemic history also played a decisive role in a coopera-

tion project of the author with Tsinghua University in Beijing. The so-called Western Buildings comprising gardens and Western style spring fountains, water features and hydraulic technology were planned by European Jesuits and executed together with Chinese architects and builders at the Old Summer Palace Yuanmingyuan in Beijing at the request of Emperor Qianlong in the mid-18th century. The emperor wanted the buildings, fountains and water features within three months. The painter Castiglione designed the architecture, the astronomer Benoist the hydraulics. In this special epistemic situation, the two Jesuits had to recover the necessary Western knowledge from the Western architecture and hydraulics treatises that were available in the Jesuit libraries in Beijing. The author investigated how the Jesuits managed to timely plan the Western buildings and spring fountains, how Chinese and Western architectural and garden concepts interacted, how the palaces and spring fountains were realised by the Chinese builders by adapting existing and imported construction methods, and how this intercultural moment was received (Schlimme, 2015; Schlimme, 2018; Schlimme, 2019).
The application of scientific methodology in construction happens in rather isolated cases until the 18th century. From the 19th century onwards standardised building techniques and the in-advance-calculation of individual constructions became ever more important. The local practical knowledge about the realisation of buildings and the experience of the craftsmen should, however, not be forgotten. Especially in the 19th and 20th centuries, local hands-on building site experience played a decisive role as a starting point, corrective and counterpart in processes of putting building knowledge on a modern scientific and industrial base. An example is the implementation of international patents such as the Hennebique's reinforced concrete construction patent (1892) on building sites in different countries, where the new construction method interacted with different structures of practical knowledge, different logistics of construction processes and design maxims. The Hennebique system had in fact to be broken down to the diverse local functioning of the building sector (Schlimme, 2009-2010).

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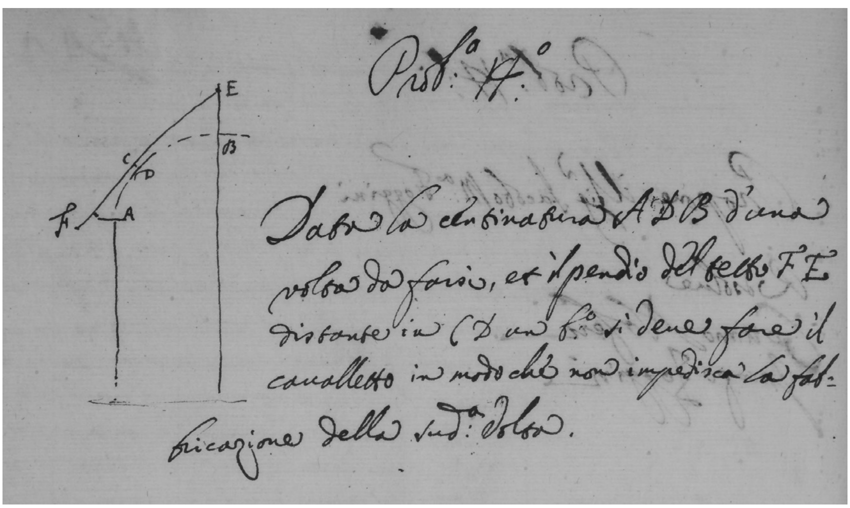
Fig. 1. Interior of San Giovannino, Florence; former Jesuit church (Bibliotheca Hertziana, photographic archive).



Prob.a 14.a

Dato la centinatura ADB d'una volta da farsi, et il pendio del tetto FE distante in CD un b.o si deve fare il cavalletto in modo che non impedisca la fabbricazione della sud.a volta.

Fig. 2. Accademia della Vachia, formulation of problem 14 with sketch (BNCF, Fondo Nazionale, ms. II.46, f. 45*v*) (Reproduced with kind permission of the MiBACT/BNCF. Reproduction or duplication by any technique whatsoever is not permitted).

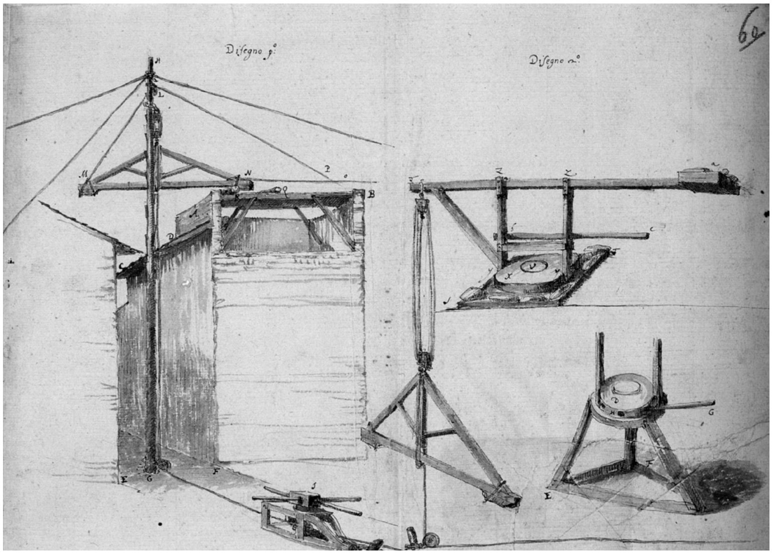


Fig. 3. Accademia della Vachia, problem 17: location of roof trusses; solution by Cosimo Noferi (BNCF, Fondo Nazionale, ms. II.46, f. 60*r*) (Reproduced with kind permission of the MiBACT/BNCF. Reproduction or duplication by any technique whatsoever is not permitted).

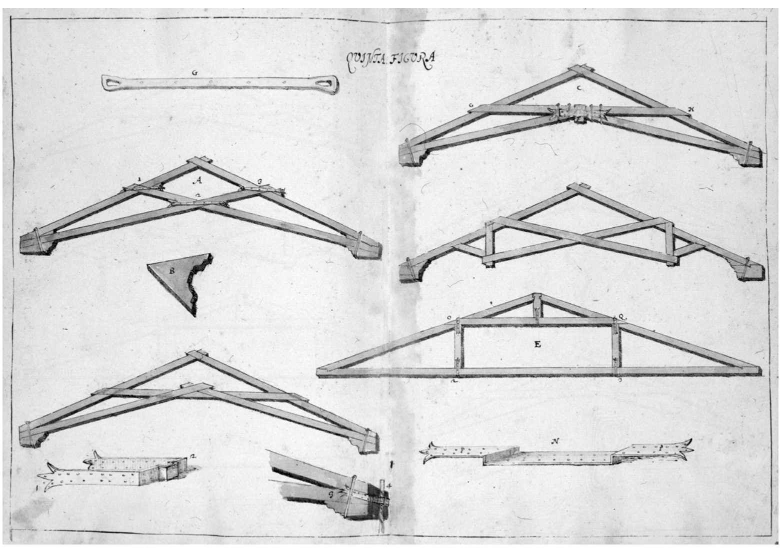


Fig. 4. Cosimo Noferi, *Tomo primo della Travagliata Architettura*, Quarto discorso, Quinta Figura (BNCF, Ms., Gal. 122) (Reproduced with kind permission of the MiBACT/BNCF. Reproduction or duplication by any technique whatsoever is not permitted).

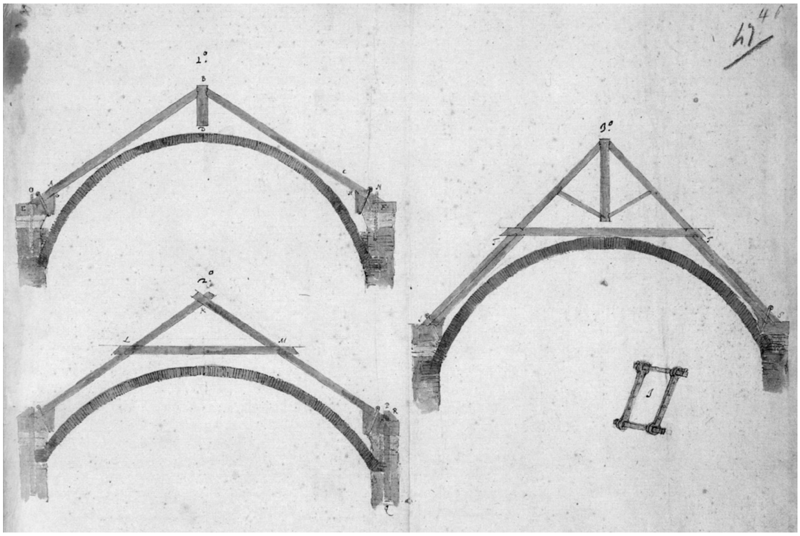


Fig. 5. Accademia della Vachia, problem 14; solution by Cosimo Noferi (BNCF, Fondo Nazionale, ms. II.46, f. 47*r*) (Reproduced with kind permission of the MiBACT/BNCF. Reproduction or duplication by any technique whatsoever is not permitted).

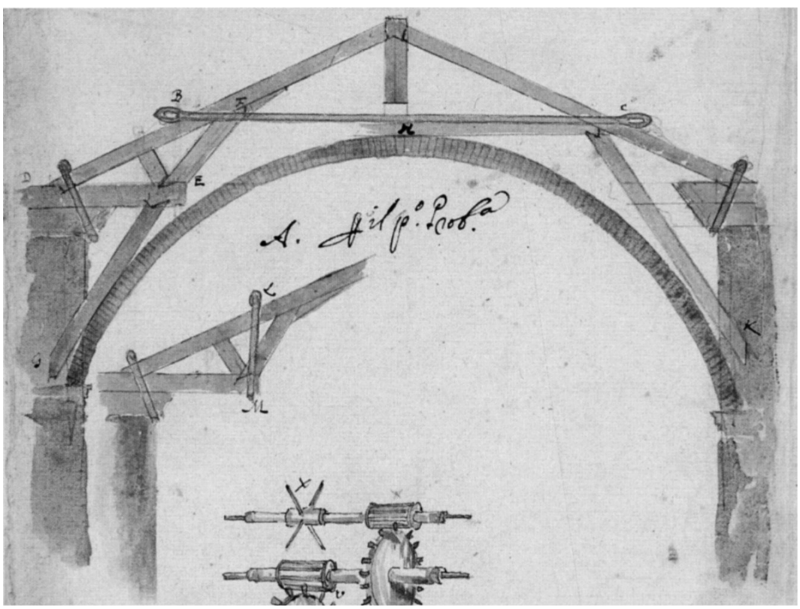


Fig. 6. Accademia della Vachia, problem 37; solution by Cosimo Noferi (BNCF, Fondo Nazionale, ms. II.46, f. 120*r*) (Reproduced with kind permission of the MiBACT/BNCF. Reproduction or duplication by any technique whatsoever is not permitted).

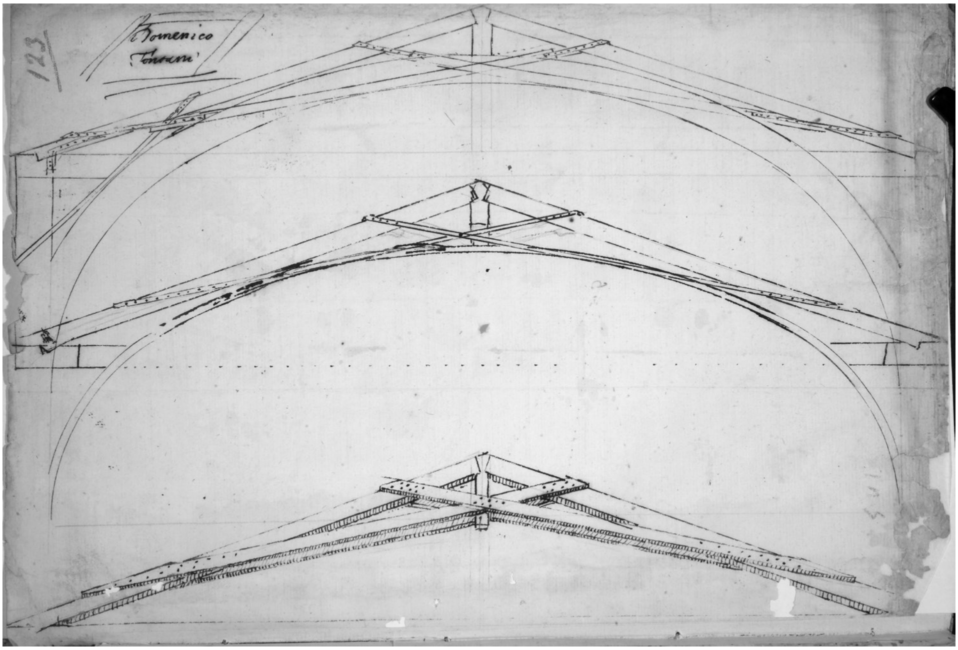


Fig. 7. Accademia della Vachia, problem 37; solution by Domenico Fontani (BNCF, Fondo Nazionale, ms. II.46, f. 123*r*) (Reproduced with kind permission of the MiBACT/ BNCF. Reproduction or duplication by any technique whatsoever is not permitted).

Fig. 8. San Giovannino, Florence: general view of the roof (photograph Hermann Schlimme/Jens Niebaum).

Luc Tamboréro

La coupe des voûtes à la française

Philibert De l'Orme (1510-1570), publie *Les Nouvelles inventions pour bien bâtir et à petits frais*[1] en 1561 et *Le premier tome de l'Architecture* en 1567[2]. Par ses écrits il divulgue les tracés des voûtes jalousement gardés par les constructeurs. Architecte du Roi, il a gravi les échelons qui ont permis son ascension au sein de la communauté des maçons. Pour devenir architecte, il est passé par l'apprentissage, pour devenir compagnon et demander à la communauté d'accéder à la «Maitrise». Cette maitrise est une épreuve de coupe des pierres codifiée par «les Massons» et régie par la Chambre royale des bâtiments[3], permet de prétendre à l'ouverture de sa propre entreprise, quand la communauté des maitres maçons de la ville l'autorise, dans le respect de règles imposées[4]. Tous les compagnons obtenant la maitrise n'ouvrent pas forcément d'entreprise, ils peuvent s'orienter vers le métier d'appareilleur, la connaissance de la coupe des pierres fait alors leur légitimité sur le chantier et ils peuvent être reconnus architectes[5]. Philibert de l'Orme propose que la coupe des pierres de l'appareilleur prenne part au projet d'architecture, que les bâtiments soient modelés par la stéréotomie, il emprunte la voie technique pour

[1] De l'Orme Philibert, *Nouvelles inventions pour bien bastir et a petits fraiz*, Paris, Frédéric Morel, 1561.

[2] De l'Orme Philibert, *Premier tome de l'architecture de Philibert de L'Orme*, Paris, Frédéric Morel, 1567.

[3] Carvais R., *La Chambre royale des Bâtiments. Juridiction professionnelle et droit de la construction à Paris sous l'Ancien Régime.* Thèse de doctorat d'État en droit, Université de Panthéon-Assas (Paris-II), 2001, 3 vols., pp. 205-616 (à paraître aux Éditions Droz, Genève).

[4] Les règles de l'ouverture d'entreprise varient en fonction des villes, de la capacité des Maitres de répondre à la demande des constructions du Roi et du peuple. L'ouverture de l'entreprise peut aussi être conditionnée par la religion pratiquée ou l'origine du compagnon. Voir: Carvais R., *La Chambre royale*, cit.

[5] A cette époque la profession n'est pas constituée, quiconque peut se déclarer architecte, La défense d'utiliser sans titre la qualité d'Architecte est prononcée par arrêt du Conseil d'Etat du roi du 7 mars 1676.

exprimer le pouvoir, l'ascendant sur le chantier. En divulguant les traits de coupe des pierres et surtout une méthode *universelle* s'appliquant à toute sorte de voûtes, invitant les lecteurs à inventer d'autres méthodes pour simplifier les tracés, de l'Orme dévoile son intention et fait de la connaissance stéréotomique un point essentiel pour l'existence d'architectes et d'ingénieurs indépendants du seul apprentissage par le chantier. Ce projet Delormien voit ainsi le jour dans la création de l'Académie Royale d'Architecture plus d'un siècle plus tard.
Comme toute science, la stéréotomie possède une histoire rythmée par des découvertes et des étapes. Le cours de Philippe De la Hire en 1688 à l'Académie royale d'Architecture marque une charnière dans cette histoire. Pour la comprendre, il est nécessaire de situer l'œuvre de De la Hire dans l'évolution de la stéréotomie.

La coupe des pierres est connue de l'appareilleur qui réalise les tracés géométriques grandeur nature des ouvrages en chantier. Ces tracés sont appelés épures, de l'épure sont extraits les panneaux et les angles qui permettent au tailleur de pierre ou au charpentier de couper la matière.

Les épures sont réalisées à partir d'un ensemble de méthodes et de techniques transmises secrètement entre appareilleurs, comme en témoigne par exemple le carnet de Villard de Honnecourt[6]. Un premier ensemble de méthodes a permis aux appareilleurs romans et gothiques de réaliser des voûtes et des charpentes très variées dans leurs formes, mais relativement peu dans leurs épures. L'objectif est de construire, il ne s'agit généralement pas d'une recherche de la vérité géométrique mais bien d'un moyen de construction. Obtenir le plus rapidement et efficacement possible les panneaux et angles de taille est le principal but de l'épure de coupe des pierres. Les travaux que nous avons présentés au sujet des Vis Saint Gilles en sont un exemple[7].

Philibert de l'Orme est le premier architecte qui divulgue les tracés géométriques de coupe des pierres et des bois dans les deux traités d'archi-

[6] Voir Erlande-Brandenburg A., Pernoud R., Gimpel J., Bechmann R., *Carnet de Villard de Honnecourt*, XIIIe siècle, Paris, Stock, 1994, fol. 21*r*, pl. 41.

[7] Voir Tamborero L., "*The Vis Saint-Gilles, Symbol of Compromise between Practice and Science*", in *Proceedings of the Second International Congress on Construction History*, Cambridge, The Construction History Society, 2006, pp. 3025-3038.

tecture qu'il publie. Il peine à rendre intelligible la description du tracé des figures. Il s'en excuse à plusieurs reprises et l'organisation du traité force le lecteur à le percevoir comme une accumulation de tracés géométriques glanés sur les chantiers; c'est certainement intentionnel, il souhaite faire connaître au public son invention en coupe des pierres: un tracé universel.

Cette invention, matérialisée dans son traité et dans son œuvre d'architecte par la trompe du château d'Anet mais aussi par les trompes de l'hôtel Bullioud à Lyon, ne réside pas dans la forme particulière des trompes mais dans le trait qu'il utilise pour parvenir à en tracer les claveaux. Ce tracé a une propriété qui vaut par-dessus toutes les autres en coupe des pierres: il est universel[8], c'est-à-dire qu'il peut être appliqué à tous les cas de figure d'une voûte conique.
En présentant cette première invention, de l'Orme dessine les contours d'un projet de recherche plus large: il propose de réduire en 6 méthodes universelles la plus grande partie des tracés géométriques pour la coupe des voûtes[9].

La mise au point de méthodes universelles est sans doute la voie la plus séduisante pour comparer les auteurs de l'histoire de la coupe des pierres et de la stéréotomie, de l'Orme est ainsi directement comparable à Desargues par le «brouillon project[10] » et la Géométrie descriptive[11] de Monge peut très bien être vue comme un objectif à atteindre. Mais réduire l'histoire de la coupe des pierres à la seule théorie et faire fi du choix des hommes, de leurs intérêts sociaux et politiques quant à la valorisation d'une méthode géométrique ou de son interdiction pour l'accès à la maitrise, pour l'accès à l'entreprise, ne permettrait pas de refléter son évolution. Nous présentons ici les bases géométriques permettant la lecture des évolutions dans les épures et dans les traités pour permettre au lecteur de les appréhender et nous présenterons ensuite les contextes et les destins de ces méthodes et inventions.

[8] Dans le cas des voutes coniques.

[9] De l'Orme Philibert, *Premier tome*, cit., p. 128.

[10] Desargues Girard, *Brouillon Project d'exemple d'une maniere universelle du S.G.D.L.* [Sieur Girard Desargues Lyonnais] *touchant la pratique du trait à preuve pour la coupe des pierres en l'Architecture* ..., Paris, 1640.

[11] Monge G., *Géométrie descriptive. Leçons données aux écoles normales, l'an 3 de la république,* Paris, Baudouin, 1798.

Nos travaux du diplôme de l'Ecole des hautes etudes en sciences sociales, récompensé par le prix Edouardo Benvenuto, démontrent l'existence de bases géométriques qui figurent dans toutes les épures. En distinguant ces bases géométriques, nous créons un outil de lecture grâce auquel tout ajout ou modification dans les épures se remarque, ce qui permet de distinguer les inventions des auteurs des méthodes précédentes[12].
Il s'agit de *la perche*, *la rotation*, *le rabattement* et *le développement*.
Ces quatre bases géométriques sont des opérations sous-jacentes à toute méthode de tracé des voûtes gothiques ou de la Renaissance. Les connaître permet de tracer et d'obtenir les panneaux et angles de taille de toutes les voûtes en pierres, de comprendre le cheminement de l'épure, sa nature.

Ces bases constituent le patrimoine géométrique qui permettait aux maçons d'asseoir leur hiérarchie et leur légitimité sur les chantiers. L'architecte des chantiers médiévaux était donc généralement un appareilleur au sommet de son art et reconnu par ses pairs et sa communauté de métier. Ce système élitiste et clos garantissait tous pouvoirs à la communauté des maçons sur les chantiers et l'affiliation de ceux qui peuvent se prétendre architectes.

Philibert de l'Orme, qui est lui-même issu d'une famille de maîtres maçons[13], propose, dans ses deux traités d'architecture, une nouvelle figure dans la hiérarchie des constructeurs de la Renaissance: l'Architecte français. Ce nouvel architecte doit selon lui exercer librement, être cultivé aux arts, aux sciences, aux techniques et, particularité française, exprimer la forme la plus subtile de son art à travers l'invention de nouvelles voûtes en pierre. De l'Orme donne l'exemple en décrivant les tracés géométriques qui lui ont permis de réaliser les voûtes sur ses chantiers. Par la description de ses épures, il devient le premier auteur à donner les tracés des voûtes au public. Son intervention ne se limite pas à diffuser un savoir jusqu'alors confidentiel, il propose surtout de moderniser les méthodes

[12] Tamboréro L., *De Delorme à la Hire, la recherche d'une méthode universelle dans les traités de stéréotomie. Opérations géométriques et emprunts*, mémoire du diplôme de l'EHESS, Directeur du mémoire: Dhombres J., Spécialité: Histoire des sciences, Paris, EHESS, 2008.

[13] Son frère est contrôleur de bâtiments du roi. Au sujet de De l'Orme, voir Potié P., *Philibert Delorme, figures de la pensée constructive*, Marseille, Editions Parenthèses, 1996.

géométriques grâce à l'apport scientifique de la géométrie euclidienne et de l'astronomie.

Sa proposition est illustrée en pratique par l'exemple des constructions qu'il a pu mettre en œuvre pendant sa carrière. Notamment l'invention d'une nouvelle méthode géométrique pour tracer les voûtes de l'hôtel Bullioud à Lyon, son premier travail d'architecte à son retour d'Italie.

De l'Orme écrit:

> J'en trouvay le traict & inventay l'artifice en ladicte année mil cinq cens trentesix, par le moyen & ayde de Geometrie, & grand travail d'esprit: lequel je n'ay plainct depuis, ains plustost louë Dieu grandement, de ce que d'un seul traict, & seule façon de trompe, on les peult faire toutes.

Les méthodes géométriques qu'il utilise sont *la rotation* et surtout *la perche*, qu'il développe en lui ajoutant une propriété géométrique du théorème de Thalès. Philippe Potié[14] observe qu'en termes de résolution géométrique ou technique, cette invention n'apporte rien de nouveau aux techniques de trait médiévales. Nous ajouterons que la méthode a pu paraître absurde aux appareilleurs contemporains de De l'Orme, car *la perche* n'est pas la méthode traditionnellement requise pour faire des trompes coniques. Ce sont normalement les méthodes du *rabattement et du développement* qui sont utilisées pour obtenir les panneaux de taille des trompes et qui diminuent considérablement le temps de tracé, comme l'indiquent les croquis de Villard de Honnecourt.
De l'Orme démontre sa connaissance *du rabattement et du développement* dans d'autres épures, ce n'est donc pas une carence technique qui suscite ses choix mais un objectif précis: la possibilité de variation de formes de la voûte.
Son invention repose sur un enseignement géométrique facile d'apprentissage et déjà acquis par les appareilleurs de son temps: *la perche* et *la rotation* sont employées par les appareilleurs dans le tracé des voûtes d'ogives. Tout le génie de De l'Orme est de parvenir à conserver la plasticité de ces méthodes pour le tracé des trompes. Il apporte aux tracés ordinaires de nouvelles potentialités en puisant dans les modèles géométriques médiévaux, préférant donner aux futurs architectes des méthodes

[14] *Ibidem.*

géométriques nécessitant certes un plus grand travail d'épure mais permettant de faire varier à l'infini les formes bâties.

> [...] du seul traict que je propose, ils pourront faire toutes sortes de trompes & de surpentes creuses par le dessous: j'entend toutes façons de voutes que vous voudrez penser, pour estre suspendues en l'air[15].

De l'Orme donne comme exemple probant de son projet la trompe d'Anet qu'il a réalisé pour le roi.
Son ambition ne s'arrête pas au seul cas des trompes, sa quête est bien la mise au point de méthodes géométriques universelles pour tracer la coupe de toutes voûtes.
Le texte de conclusion de la coupe des pierres de De l'Orme peut être vu comme son testament d'appareilleur:

> L'Architecte doit sçavoir une infinité de ces sortes de traicts [...]. (Mais on pourrait les) abreger par le moyen de l'intelligence & connoissance de la nature de six sortes de traicts ou figures Geometriques extraictes d'Euclide & Archimèdes. La premiere sorte servira pour toutes les descentes & voutes de caves [...], L'autre servira pour sçavoir toutes sortes d'arches & portes. La troisième pour toutes trompes. la quatrieme pour toutes sortes de voutes sphériques [...] la cinquième pour toutes façons d'escaliers, la sixieme pour toutes sortes de vis. Si quelqu'un les peuve trouver, ils seront cause d'un grand repos & soulagement pour moi.

De l'Orme a manifestement mis au point la troisième «sorte de trait», qui sert au tracé de toutes trompes, et laisse un projet pour ses successeurs: découvrir d'autres méthodes et s'affranchir du secret par la science.

Deux manuscrits d'appareilleurs français nous sont parvenus après les publications de De l'Orme, dont les auteurs n'adhèrent visiblement pas aux propositions de l'Architecte du Roi. Jean Chéreau[16] et Jacques Gentilhâtre[17] n'ont que faire des recherches architecturales de De l'Orme. Chéreau en

[15] De l'Orme Philibert, *Premier tome*, cit., livre IV, chap. II, fol. 91.

[16] Chéreau Jean, *Livre d'architecture*, 1567-1574 (selon Jean-Marie Pérouse de Montclos), Gdansk, Bibliothèque Municipale, ms. 2280.

[17] Gentilhâtre Jacques, *Manuel d'architecture attribué à J. Gentilhâtre par L. Châtelet-Lange*, ms., 1620-1630, Bibliothèque Nationale, fonds français, 14.727.

particulier s'applique à recopier le plan des épures de De l'Orme, pour les corriger selon les méthodes géométriques traditionnelles appelées «la traditive». Les trompes y sont tracées avec les méthodes de *rabattement et développement* et la trompe d'Anet n'y est même pas mentionnée.
Les deux manuscrits témoignent en revanche de l'intérêt des auteurs pour une application scientifique de l'Astronomie: la Gnomonique. Cette science géométrique permet de tracer les mouvements apparents du soleil sur la terre grâce à l'ombre portée d'une tige (appelée style) plantée sur le mur d'une maison, ou d'un objet quelconque. Les mouvements des astres et l'ensemble des données nécessaires au tracé des épures sont symbolisés par des plans, des axes et des angles.
Chronologiquement contemporaine des ouvrages de De l'Orme, la vulgarisation de la gnomonique par sa traduction en Français commence en 1556 avec six ouvrages publiés en 1569: cinq à Paris, un à Poitiers.

- Claude de Boissière Daulphinoys, *La propriété et usage des quadrans nouvellement exposée*, Paris, 1556, réédité en 1567, In 8°, 31 pages.
- Jean Bullant, *Recueil d'horlogiographie, qui contient la description, fabrication et usage des horloges solaires,* Paris, 1561[18], In 4°, 142 pages.
- Jean Bullant, *Petit traicté de géométrie et d'horlogiographie pratique,* Paris, 1562, In 4°, 28 pages.
- Elie Vinet*, La Manière de faire les solaires, que communément on appelle cadrans,* Poitiers, 1564, In 4°, réédité en 1583 et 1607.
- Pierre Forcadel*, La description d'un anneau horaire,* Paris 1568, In 8°, 16 pages.
- Pierre Forcadel*, La description d'un anneau solaire convexe,* Paris 1569, In 4°[19].

[18] Nous avons consulté un manuscrit avec le titre *Des orloges solaires et ombratils de Bullant architecte de Mr de Montmorency.* Bibliothèque de l'Arsenal, Paris, ms. III 147 2890, f°s 318r-359. Les nombreuses ratures et les nombreuses réécritures du texte permettent de penser qu'il s'agit du brouillon original du *Recueil d'horlogiographie* de 1561. Dans ce manuscrit, certains croquis en marge du texte témoignent des réflexions de Bullant sur les épures auxquelles il travaillait.

[19] Gotteland A., *Les cadrans solaires et méridiennes disparus de Paris*, Paris, CNRS, 2002, p. 33.

Les traités précédents écrits en latin[20] sont souvent la source de ces nouveaux ouvrages comme en témoigne Jean Bullant, architecte royal, dans son avis au lecteur[21]. L'usage de la langue française facilite l'accès aux connaissances et l'exemple de Bullant a certainement encouragé les architectes et les ouvriers de son temps à étudier les plans et les volumes à travers la gnomonique.
En gnomonique comme en perspective, les outils géométriques utilisés sont *le rabattement* et *le changement de plan de projection*, mais la gnomonique présente des applications beaucoup plus proches de la coupe des pierres, notamment dans les travaux de Girard Desargues en partie très comparables aux tracés des cadrans solaires déclinants, alors que la perspective a, selon nous, une contribution bien inférieure dans l'histoire de la coupe des pierres[22].

Vers 1640, les travaux contemporains de deux scientifiques, Girard Desargues et Florimond de Beaune, permettent d'envisager l'héritage des architectes royaux Bullant et de l'Orme sous un nouveau jour. Deux ouvrages, l'un publié, l'autre manuscrit, utilisent la géométrie des cadrans solaires déclinants pour donner un nouvel outil à l'architecture et à la coupe des pierres, c'est la *Méthode Générale* selon l'appellation de Philippe de la

[20] Trois traités surtout sont remarquables:

- Finé Oronce, "De solaribus horologius ab quadrantibus. Libri IIII [1531]", *in Protomathesis. Opus nunc primum in lucem emissus* [...], Paris, 1532, réédité en 1560.

- Münster Sedastian, *Compositio horologiorum, in plano, muro, truncis, anuto, concavo, cylindro et quadrantibus*, Bâle, 1531.

- Münster Sebastian, *Horologiographia*, Bâle, 1533. Ce dernier ouvrage contient une première partie consacrée à la géométrie utile aux problèmes de la gnomonique; Bullant copiera cette forme de présentation dans son second traité.

[21] Bullant dans son premier traité indique s'être inspiré des ouvrages en latin d'Oronce Finé et Sebastian Münster.

[22] La perspective rend compte d'un espace plan déformé par un point de vue. La gnomonique prend toujours en compte plusieurs plans de projection et permet une approche volumique en tenant compte de l'inclinaisons de la terre par rapport au soleil, de l'inclinaison de la façade sur la terre et de son angle par rapport aux méridiens. Enfin la similitude entre le plan des heures et le plan des joints d'une arche renforce encore les similitudes géométriques.

Hire[23] en référence à la description de la «préparation Générale[24]» dans le texte d'Abraham Bosse **[fig. 1]**.

Girard Desargues publie Le *Brouillon Project d'exemple d'une maniere universelle touchant la practique du trait à preuve pour la coupe des pierres en l'Architecture* (1640). Il cherche à répondre aux attentes des constructeurs et des architectes du XVII[e] siècle. Le titre contient la notion d'universalité, mais aussi de complémentarité et de vérification du tracé et des angles ou panneaux d'où l'appellation de «trait à preuve».

Presque en même temps, Florimond de Beaune, sans doute inspiré par la gnomonique, rédige la *Doctrine de l'Angle Solide contenu sous trois angles Plans*[25] (1638-39 selon Pierre Costabel), manuscrit conservé aux archives de l'Académie des Sciences de Paris[26], dans lequel il précise que «personne n'a encore rien fait [de manière purement géométrique] touchant l'angle solide […] quoique […] l'usage en soit fort utile à l'architecture»[27].

Les deux travaux sont différents mais utilisent le même procédé géométrique dans les rabattements de plans où les notions de pesanteur induite par la vue en plan et l'élévation disparaissent au profit d'une focalisa-

[23] La Hire Philippe de, La *Couppe des Pierres*, Paris, Ecole Nationale des Ponts et Chaussés, ms. 228, t. I, 1688-1690, t. II, 1702-1704. Voir t. I, fol. 72-73.

[24] Bosse Abraham, *La pratique du trait a preuves de mr desargues lyonnois Pour la Coupe des Pierres en l'Architecture*, Paris, des Hayes, 1643, p. 38: «la *pratique* des trouver cet arc droit ou beveau de deux paneaux est si *générale*» et p. 45-46: «Préparation Générale» tables des planches 12 à 25.

[25] Beaune Florimond de, *Doctrine de l'angle solide,* [1638-1639 selon Costabel Pierre]. Transcription du manuscrit voir Costabel P., Barbiche B., *Florimond de Beaune, Doctrine de l'angle solide, inventaire de sa bibliothèque*, Paris, Vrin, «collection des travaux de l'Académie internationale d'histoire des sciences», n. 19, 1975.

[26] Merci à Guy Picolet qui nous a appris lors du colloque Philippe De la Hire à l'Académie d'Architecture de Paris en 2010 l'existence de ce manuscrit dans le bagage de l'Abbé Picard (1620-1682) à son retour du Danemark. Le manuscrit est donc passé dans les mains de Philippe de La Hire à la mort de Roberval à qui il succède au Collège de France ou directement de Picard à de La Hire par leurs liens directs comme le suggère Guy Picolet. Voir Guy P., "La Hire, ami, collègue, éditeur et continuateur de Picard**"**, in Becchi A., Rousteau-Chambon H., Sakarovitch J. (dir.), *Philippe de La Hire (1640-1718) entre architecture et sciences*, Paris, Picard, 2013.

[27] Costabel P., Barbiche B., *Florimond de Beaune*, cit.: Introduction de Pierre Costabel, p. 20.

tion sur les angles dièdres et les faces de l'objet dessiné. Les auteurs démontrent dans leurs essais que les corrélations entre les angles dièdres et les angles des plans sont suffisants pour définir les volumes à l'aide d'une fausse équerre. Florimond de Beaune explore généralement les corollaires de sa «doctrine»; Desargues utilise les mêmes rabattements de plan pour accéder aux «essieu » du *Brouillon Project*. Il emprunte beaucoup de termes à la gnomonique et s'efforce de rendre intelligible chaque étape de ses tracés. «Le style» est appelé «essieu», la ligne traversière d'un cadran solaire déclinant est appelée traversieu (traversière + essieu).

Si on peut affirmer que l'invention de Desargues commence à partir du tracé des «essieu» et lui appartient pleinement, étant une invention complète, ce n'est pas le cas du tracé préliminaire qu'il a en commun avec le premier tracé du manuscrit de Florimond de Beaune, la Doctrine de l'angle solide, celui-là même que Philippe de La Hire appelle «*Méthode Générale*».

Le tracé existe en Gnomonique, et s'applique aux cadrans solaires déclinants. On le trouve imprimé par Oronce Fine en 1531 et vulgarisé par Bullant en 1561.

Desargues souhaite probablement développer le potentiel universel de la *Méthode Générale*, le père Mersenne annonce en effet en 1644 la future publication de nouveaux traités arguésiens, dont un sur l'angle solide[28] malheureusement inconnu aujourd'hui.

Mathurin Jousse publie en 1641 son traité de coupe des pierres[29] suivi par le Père Derand[30] deux ans après. Ces traités rassemblent un grand nombre de tracés mais n'apportent pas de réelle nouveauté, si ce n'est dans l'explication du tracé des figures et leur multitude. Le père Derand défend une science principalement orientée vers un traitement typologique de la forme des voûtes. La trompe d'Anet de Philibert de l'Orme devient par exemple une trompe ondée et rampante.

[28] Mersenne M., *Cogitata physico-mathematica*…, Paris, 1644, *Hydraulica pneumatica…, Prœfatio ad lectorem*, p. 11, non paginée. Cf. *supra* et note 55.

[29] Mathurin Jousse, *Le Secret d'architecture*, La Flèche, 1642.

[30] Derand François, *L'Architecture des voûtes ou l'art des traits et coupe des voûtes*…, Paris, 1643.

La recherche de méthodes universelles ne fait toujours pas partie des préoccupations de Derand ou de Jousse.

Mathurin Jousse est un rapporteur d'épures, son traité est un recueil de planches issues de la tradition, et de secrets de maitre d'œuvres comme il aime à nommer son ouvrage, *Le Secret d'Architecture.* C'est grâce aux explications de Jousse que l'on peut expliquer l'épure de la vis Saint Gilles que de l'Orme tient secrète dans son traité d'Architecture.
Les épures issues de la tradition n'ont pas pour objectif d'approcher une vérité géométrique, mais bien de réaliser un ouvrage et Jousse s'en tient à cela.
Le Père Derand fourni, en revanche, un traité qui, sans renoncer à l'approximation géométrique pour maintenir la faisabilité des ouvrages, tend à la théorisation de la coupe des pierres par l'organisation du traité, la qualité du texte et des gravures.

Le manuscrit de Florimond de Beaune, la publication du *Brouillon Project* de Desargues, ainsi que le traité de coupe des pierres de Desargues par Abraham Bosse, *La pratique du trait a preuves…*[31] en 1643, sont très différents des ouvrages de Derand et de Jousse. La méthodologie utilisée par les auteurs pour la description des épures, passe en premier lieu par le fondement géométrique de la théorie. La méthode géométrique est décrite et démontrée avant d'être appliquée aux différentes épures d'ouvrages de coupe des pierres. Les ambitions de Desargues et Bosse sont de proposer un procédé scientifique en opposition à «la vieille manière de Trait»[32], cette nouvelle méthode se doit d'être infaillible, générale, universelle[33].

Desargues est guidé par la raison dans son intention de divulguer ses inventions au peuple. Il entreprend un cours privé pour les artisans et artistes. Il n'est pas membre de la communauté des constructeurs, c'est un scientifique, «un Géomètre contemplatif» pour reprendre ses termes. La publication de son invention suivie d'un traité est perçue par les appareilleurs comme une menace à la structure hiérarchique des bâtisseurs. Desargues est attaqué par la «communauté des Massons» en la personne de Jacques Curabelle, appareilleur à la chapelle de la Sorbonne et vraisem-

[31] Desargues Girard, *Brouillon Project*, cit.

[32] Bosse Abraham, *La pratique du trait*, cit., p. 114.

[33] *Ivi*, p. 38.

blablement d'une culture géométrique considérable. *Les examens du sieur Desargues*[34] par Curabelle est un document très intéressant[35] dans lequel l'auteur fait explicitement référence à la gnomonique et «aux déclinants»[36] comme principale inspiration de Desargues et conteste tout apport aux tracés existant déjà en Gnomonique.
Curabelle est le gendre de Jean Thiriot[37], architecte et ingénieur des bâtiments du Roi anobli par le Cardinal de Richelieu pour les services rendus contre les protestants à la Rochelle. Desargues, «qui avait eu quelque part à tous ces dessins»[38], servait comme ingénieur à la Rochelle[39] où Descartes lui rendit visite. Il est donc plausible que Desargues et Thiriot se connaissent mais aucune source documentée ne l'affirme à notre connaissance.

Entre 1635 et 1642, Thiriot est l'entrepreneur du chantier de la Chapelle du collège de la Sorbonne commandé par Richelieu, et son gendre en est l'appareilleur[40]. Curabelle se rend certainement compte de l'invention de Desargues mais la passe volontairement sous silence et s'engage dans un échange de pamphlets, de provocations publiques et l'invite au duel technique.
Desargues exprime alors son inconfort face à l'oppression qu'il subit par la communauté des maçons de Paris:

[34] Curabelle Jacques, *Examens des œuvres du Sieur Desargues*, Paris, 1644.

[35] C'est Curabelle qui donne le nom de *Stéréotomie* à la science de la coupe des pierres. Voir *Ibidem* dans les privilèges et plusieurs fois dans le texte, Curabelle analyse aussi le traité de Derand avec la même verve utilisée contre Desargues.

[36] *Ivi,* p. 10.

[37] Voir Baillot de Ligny M., "Jean Thiriot", in *Chroniques Bavaroise*, Bar le Duc, 1847, pp. 227-264. L'auteur déclare retranscrire les lettres de Thiriot à son frère, mais le texte comporte des incohérences comme la publication de *l'Architecture du Père Derand* en 1617, dont la première édition est en fait de 1643.
De nombreux documents d'archives existent à propos des Thiriot et Curabelle, notamment aux Archives de l'Aube, cote IE De Noel 20 à 22.

[38] Selon Baillet Adrien (1649-1706), *La vie de Monsieur Descartes*, Paris, Hortemels, 1691.

[39] Voir Poudra N.G., *Œuvres de Desargues réunies et analysées*, Paris, Leiber, 1864, t. I, p. 13.

[40] Voir Le Blant, "L'ancienne Sorbonne et son église d'après des documents inédits du Minutier central à Paris", in *Société historique et archéologique du Ve arrondissement. Bulletins ronéotypés*, 1974, n. 171, pp. 46-56.

> Moi, je ne suis artisan de la main d'aucune sorte d'Art; je n'ai que la simple connaissance de la raison de l'effet des règles du peu de traits dont je propose la pratique; et je n'ai point une cabale d'ouvriers comme il a ses compagnons [...]; par le nombre et par la crierie desquels il voudrait m'opprimer; et comme il sait que je ne travaille point de la main, il me voudrait engager à quelque travail effectif de maçonnerie ou bien à dépendre de la discrétion des ouvriers, ses compagnons de coterie; ce qui ne serait ni juste ni raisonnable[41].

Desargues relate dans cet extrait sa confrontation au groupe des compagnons maçons qui entourent Curabelle, le témoignage est important pour l'histoire de la coupe des pierres car il reflète la cohésion d'opinion qui permet ou non l'assimilation des techniques et des tracés en fonction de leur contexte social. Nous supposons que l'acceptation de la méthode arguésienne par la communauté des maçons de Paris est compromise quand un appareilleur comme Curabelle, farouchement opposé à son étude, entraine les compagnons dans une croisade contre son créateur.

La «communauté des Massons de Paris» bénéficie d'un pouvoir réglementaire à travers *la Chambre des Bâtiments*[42] en la «Communauté des Jurez du Roy, es Œuvres de Massonerie de la Prevosté et Vicomté de Paris»[43], appareil juridique contrôlant tout ce qui touche à la construction, de la qualité des constructions à la formation des maitres par les épreuves de la maitrise en l'art de maçonnerie et disposant d'une police.

Dans sa reconnaissance du traité de la coupe des Pierres d'Abraham Bosse[44], Desargues rapporte un extrait du Registre de cette juridiction concernant la réception à la Maitrise de Charles Bressy, maçon à Paris, en septembre 1642 qui prouve que les jurés ne se sont pas officiellement prononcés contre les méthodes de Desargues pour la réalisation du Chef d'œuvre[45].

[41] Desargues Girard, *Récit au vray de ce qui a esté la cause de faire cet escrit*, Paris, 1644.

[42] Voir: Carvais R., "Creating a Legal Field: Building Customs and Norms in Modern French", dans Kurrer K.-E., Lorenz W., Wetzk V. (dir.), *Proceedings of the Third International Congress on Construction History*, Cottbus, Brandenburg University of Technology, 2009, pp. 321-328.

[43] Voir Bosse A., *La pratique du trait*, cit., p. 54.

[44] *Ibidem.*

[45] Pour le déroulement des épreuves de la maitrise, voir Carvais R., *La Chambre royale*,

Il assure ainsi la protection de son œuvre, si sa technique avait été classée hors norme par la chambre des bâtiments elle serait devenue sans intérêt pour la Maitrise et vouée à disparaitre.
Les jurés étant issus de la communauté des maçons, l'utilisation de méthodes arguésiennes pour la maitrise reste cependant assez improbable.
La communauté des gens de métiers n'est pas uniquement composée de maçons parisiens.
Les différences de religions ou d'origine peuvent s'avérer être un problème pour faire partie de la communauté des maitres de la ville, accéder à la maitrise et à l'entreprise. Certains quartiers parisiens comme l'Enclos du Temple ou Saint-Jean-de-Latran possèdent leur propre juridiction et permettent la libre entreprise[46], mais la plupart des étrangers et protestants œuvrant sur les chantiers sont soumis aux règles de la communauté des maitres maçons de Paris et à la Police des bâtiments qui en contrôle le monopole.

Il existe une organisation de compagnons liés entre eux par un serment dit du Devoir mais coexistant secrètement avec le reste de la communauté de métier de la ville. Leur objectif est de contrebalancer la soumission à l'organisation communautaire officielle et à la juridiction ordinaire en favorisant leur accès aux postes dirigeants de la communauté grâce au nombre de voix lors des élections des syndics et jurés. L'existence de ce mouvement «présyndical», appelé le Compagnonnage du Devoir, est révélée dans la décennie 1640 par la persécution qu'il subit de la part de la Compagnie du Saint Sacrement, société secrète Catholique[47], et qui s'exerce sur les métiers de «cordonnier[48], mais aussi chapeliers, tailleurs d'habit, et selliers[49]».

cit., p. 1139.

[46] Delpeuch E., "Les marchands et artisans suivant la cour**"**, in *Revue historique de droit français et étranger*, Quatrième série, vol. 52, (juillet-septembre 1974), n. 3, pp. 379-413.

[47] Voir Allier R., "Le compagnonnage**"**, in *La Cabale des Dévots*, Paris, Librairie Armand Colin, 1902, p. 193.

[48] *L'Artisan chrestien, ou la Vie du bon Henry, maistre cordonnier à Paris, instituteur et supérieur des frères cordonniers et tailleurs*, par J.-Ant. Vachet, prestre, Paris, Desprez, 1670. Le livre témoigne des œuvres d'Henri Michel Busch (1608-1666) fondateur d'un mouvement catholique et anti compagnon du devoir.

[49] Lebrun P., *Superstitions anciennes et modernes, prejuges vulgaires qui ont induit les Peuples a des usages et a des pratiques contraires a la Religion ... avec des figures qui*

La censure émise par la Sorbonne le 20 septembre 1645[50] dénonce et condamne les pratiques rituelles du Compagnonnage du Devoir chez les Cordonniers. Raoul Allier[51] établit les liens de l'enquête menée par La Compagnie du Saint Sacrement[52], qui peuvent être complétées par les Edits de la Juridiction ecclésiastique[53]. Cette insoumission professionnelle et spirituelle, combattue par les jurés des communautés et par les instances ecclésiastiques est documentée[54]. Les compagnons du Devoir sont notamment condamnés pour faire société indépendamment de leur religion, catholique ou protestante, ce qui justement leur permettra de se développer en dehors de la volonté d'exclusion[55], très présente bien avant la révocation de l'édit de Nantes[56]. Les actes d'accusation permettent d'appréhender le bouleversement en cours dans l'organisation corporative et le basculement du compagnonnage ordinaire vers le compagnonnage du Devoir.

Pour l'histoire des sciences il peut être important de comprendre comment des individus détenteurs d'une connaissance technique, d'un métier, se regroupent en sociétés et sont amenés à modifier les rapports au pouvoir de l'institution ordinaire qui règle la vie de la communauté de la ville et la délivrance de la maitrise: les métiers de cordonnier, chapeliers, tailleurs d'habit, et selliers semblent être les seuls concernés en 1645. En 1640, rien n'indique que la communauté des maçons soit sujette à une lutte interne entre les compagnons du Devoir et les Maitres maçons de Paris quand Curabelle attaque Desargues.
Les actes corporatifs de la chambre des bâtiments rapportent des séances d'élection où les juges tentent l'éviction des «Cabalistes»[57] en 1700.

represent ces pratiques, Paris, librairie orientale de Dondey-Dupré Père et Fils, 1736, pp. 12-14.

[50] *Ivi*, p. 13

[51] Allier R., "Le compagnonnage**"**, cit.

[52] Le Vachet J.-A., *L'Artisan chrestien*, cit.

[53] *"*Edits du roi concernant la juridiction ecclésiastique*",* in *Nouvelle Pratique Bénéficiale et ecclésiastique introduite par les édits du roi*, Paris, Pralard fils, 1703, pp. 288-296.

[54] *Ibidem.*

[55] Thillay A., *Le faubourg Saint-Antoine et ses "faux ouvriers": La liberté du travail à Paris aux XVIIe et XVIIIe siècles,* Joël Cornette, Seyssel, Champ Vallon, 2002. p. 174.

[56] Voir Le Vachet J.-A., *L'Artisan chrestien*, cit.

[57] Voir Carvais R., *La Chambre royale*, cit., "Acte corporatif de la sceance mémorable du 4

La parenté entre Curabelle et Thiriot[58], ainsi que leur rôle pour les chantiers du Cardinal de Richelieu et de la Sorbonne rendent aussi peu probable leur appartenance aux compagnons du Devoir, sous enquête de La Compagnie du Saint Sacrement précisément entre 1635 et 1645 et finalement considérés par la Sorbonne comme hérétiques. Les Archives[59] de la Chambre des Bâtiments ne témoignent pas non plus de troubles provoqués par les «Cabales» de Devoir dont les buts sont sans doute contraires aux intérêts d'entrepreneurs comme Thiriot et Curabelle.

Lorsque Desargues écrit «je n'ai point une cabale d'ouvriers comme [Curabelle] a ses compagnons [...]; par le nombre et par la crierie desquels il voudrait m'opprimer; et comme il sait que je ne travaille point de la main, il me voudrait engager à quelque travail effectif de maçonnerie ou bien à dépendre de la discrétion des ouvriers, ses compagnons de coterie; ce qui ne serait ni juste ni raisonnable»[60], il ne fait selon nous aucune référence aux troubles en cours avec les compagnons du Devoir mais relate de ses positions radicales dans la conception qu'il promeut des *ars mecanicae* et les *ars liberalis*.

Le compagnonnage du Devoir se développera cependant rapidement chez des maçons. Des témoignages matériels comme les blasons avec nom de compagnons passants du Devoir gravés dans la vis Saint Gilles de l'abbatiale à Saint Gilles du Gard sont les plus anciennes traces physiques connues et sont gravés à partir de 1648. Il est envisageable que la persécution subie par les compagnons du Devoir à partir de 1635 par la Compagnie du Saint Sacrement à Paris et ailleurs, les ait poussés à adopter une organisation plus itinérante, donnant lieu à leur qualité de compagnons passants du Devoir.
Les Archives du chantier de l'Hôtel de Ville d'Arles, projet de Jules Hardouin Mansart, nous ont permis d'établir la présence en 1675 d'un compa-

septembre 1703**"** (A.N.Z^{ij} 210), p. 1143.

58 Les archives des familles Thiriot Curabelle sont conservées aux archives de L'Aube. Cote IE De Noel 20 à 22.

59 Les archives de la Chambre de la Maçonnerie ne debute qu'en 1670, voir Carvais R., *La Chambre royale*, cit., p. 1143.

60 Desargues Girard, *Récit au vray de ce qui a esté la cause de faire cet escrit*, Paris, 1644.

gnon du Devoir[61]: François Chemur dit «Poytevin la Franchise» [62], ouvrier le mieux payé du chantier.
La présence de son nom de compagnon[63] dans des archives communales et le procès-verbal de la séance du 28 Juillet 1684 à l'Académie d'Architecture où Jules Hardouin Mansart, mentionne la présence d'un «Compagnon Passant venant d'Italie» sur le chantier[64], permet de rendre compte de la tolérance accordée aux compagnons passants du Devoir sur les chantiers et de l'appréciation de leur habilité sur des chantiers complexes comme la voûte de l'hôtel de ville d'Arles.

La mention d'un Compagnon Passant par Jules Hardouin Mansart est unique dans toutes les séances de l'Académie.
Mansart lors de son passage à Arles choisit pour diriger le chantier Jacques Peitret, maitre Graveur, comme Abraham Bosse, en lieu et place des architectes-massons qui se présentent. Nous avons démontré que la coupe des pierres de la voûte de l'hôtel de ville d'Arles n'est réalisable qu'en utilisant la «*Méthode Générale*», et que Jules Hardouin Mansart en a personnellement donné le trait à Béziers où Peitret l'y a rejoint.
Est-il possible que le compagnon Passant soit remercié parce qu'il conduit les tracés selon les principes de la *Méthode Générale sous* la direction de

[61] Sur le compagnonnage du Devoir, voir Icher F., *Le Compagnonnage en France au XXe siècle, histoire, mémoire, représentations*, thèse de doctorat en histoire, Université de Toulouse, 2 juin 1997. Sur le compagnonnage en Provence, voir: Bastard L., Mathonière J.-M., *Travail et Honneur. Les Compagnons Passants Tailleurs de Pierre en Avignon aux XVIII^e^ et XIX^e^ siècles*, Dieulefit, La Nef de Salomon, 1996.

[62] Registres des Délibérations et collection des Journaux Officiels, Archives communales d'Arles, (désormais ACA, ICC 693, f°1-50).

[63] On pourrait retrouver à Paris des compagnons du Devoir et leur nom, pour mettre en valeur leur présence au sein des communautés avant que celles-ci soient entièrement vouées au Devoir. Par exemple, le rôle compagnonnique le plus ancien conservé par les Honnêtes Compagnons Passant Tailleur de Pierre du Devoir de Paris date de 1720.

[64] Procès-Verbaux de l'Académie Royale d'Architecture 1671-1793, Séance Du 28 Juillet 1684: «La Compagnie a parlé de plusieurs ouvrages et entre autres de la construction d'une voûte que M. Mansart a ordonné de faire à l'hostel de ville d'Arles, dont la largeur est d'environ 7 thoises, laquelle n'a que 5 pieds de montée qui est environ un huitième de sa largeur, et qui a été construite par un Compagnon Passant, venant d'Italie, nonobstant la difficulté qui paraissait à cause du peu de montée qu'elle a et du peu d'espaisseur des murs de face, en ayant augmenté la force par quatre trompes aux quatre angles et colonnes isolées en dedans, ce que la compagnie a fort approuvé».

Peitret? Cela reste une hypothèse qui mérite d'être envisagée, la voûte d'Arles est la première du genre, les arêtes des voûtes y sont des arcs réguliers et non le résultat de pénétrations de volumes, Jules Hardouin Mansart établit volontairement les conditions pour que l'Architecte dicte le maçon, les compagnons passants du Devoir auraient tout intérêt à se prêter au jeu aux dépends des compagnons ordinaires.

Avec la création par Colbert de l'Académie Royale d'Architecture en décembre 1671, au sein de laquelle les architectes du Roi sont formés, l'institution du métier d'architecte du Roi est décrétée[65] et la seule voie qu'un appareilleur puisse emprunter pour devenir architecte du Roi devient celle de l'Académie. En revanche, le processus qui permettra de soumettre la communauté des maçons et la Chambre des Bâtiments à l'enseignement de l'Académie s'inscrit dans le temps[66]. Procès, corruption, et ouverture aux jurés bourgeois non affiliés à la communauté, auront raison du monopole corporatif[67].

François Blondel est le premier directeur de L'Académie Royale d'Architecture en 1672, il commente la seconde édition de *l'Architecture françoise* de Savot, plusieurs de ses commentaires parlent de la coupe des pierres. Blondel est surpris que «la Règle universelle de Monsieur Desargues expliquée dans le livre du Sieur Bosse, soit si peu en usage, vu qu'elle est infaillible dans la pratique et qu'elle peut servir à tous les cas»[68], contrairement aux livres de Jousse et Derand qui «contiennent l'un et l'autre autant de pratiques diverses qu'il se propose de cas différents, et qu'il y a plusieurs

[65] «Arrêt du conseil d'Etat du Roy, qui fait deffenses aux Me Maçon entrepreneurs des bât. de se dire et qualifier Architectes de Roy, s'ils ne sont reçus à l'Académie d'Architecture, 7 mars 1676. Signé Colbert». Voir Archive de la préfecture de Police, Collection Lamoignon, vol. XVI, fol. 517-518. Et Traité de la police, t. IV, p. 72, in Carvais R., *La Chambre royale*, cit., p. 1280.

[66] Voir Mignot C., Babelon J.P., *François Mansart le Génie de l'Architecture*, Gallimard, Paris, 1988, pp. 107-128; Assegont C., *Socialisation du savoir, socialisation du regard. Les usages techniques et sociaux du savoir géométrique et de la stéréotomie chez les compagnons tailleurs de pierre*, Tours, Université François Rabelais, thèse de doctorat en sociologie, 2002, p. 176.

[67] Voir Carvais R., *La Chambre royale*, cit.

[68] Dans la séance du 7 avril 1672, consacrée à Philibert de l'Orme, le «trait a preuve» est mentionné comme inventé par de l'Orme, «premier des architecte françois», la notion de trait a preuve est une invention de Desargues. acad. arch. T. 1, page 11.

de ces pratiques qui dans la rigueur de la géométrie sont fausses»[69]. Jules Hardouin Mansart a probablement suivi ces conseils pour réaliser le projet de la voûte d'Arles et la géométrie qui y est nécessaire s'inscrit dans son temps. En 1686 le décès de Blondel prive l'Académie de son professeur d'architecture, Philippe de la Hire le remplace en 1687 et introduit très rapidement la question stéréotomique à l'académie pendant les six séances de mi-octobre à mi-novembre 1688. De la Hire doit faire face aux attaques de Pierre Bullet, ancien appareilleur de Blondel, sur des questions de coupe des pierres sur les voûtes ovales. Le 12 novembre la Compagnie donne raison à De la Hire et c'est durant cette dernière séance qu'il présente l'affiche de son cours de stéréotomie:

> M. de la Hire a présenté à la Compagnie l'affiche pour les leçons d'architecture de l'année prochaine, où il promet d'expliquer le trait de la coupe des pierres, suivant la méthode ordinaire, et d'autres particularitez qu'il donnera

De la méthode ordinaire, sous entendue celle de De l'Orme, Derand et Jousse, Philippe De la Hire n'enseigne que très peu de chose, il concentre tout son enseignement autour de la «*Méthode Générale*», la géométrie de l'angle solide de Florimond de Beaune, la préparation au génie de Desargues.

La Doctrine de l'Angle Solide de De Beaune fait partie des ouvrages que De la Hire est chargé de mettre en ordre dans le rangement de la bibliothèque de l'Académie d'Architecture. «Picard ait montré relativement tôt à La Hire le manuscrit de l'ouvrage de Florimond de Beaune intitulé «La doctrine de l'angle solide contenu sous trois angles plans» qu'à son retour du Danemark au printemps 1672, Érasme Bartholin lui avait confié aux fins d'édition en France». Ce manuscrit n'est revenu du Danemark à Paris qu'au début de l'année 1673[70] pour être remis à l'abbé Picard (1620-1682) en vue de sa publication.
Dans l'édition de *la Doctrine de l'Angle Solide*, Pierre Costabel signale que «le traité de l'angle solide de De Beaune était tombé entre les mains de Roberval […] et passé avec la succession de Roberval dans les Archives

[69] Note de Blondel dans Savot Louis, *L'architecture françoise des bastimens particuliers*, seconde édition, Paris, 1673, p. 352.

[70] Beaune Florimond de, *Doctrine de l'angle*, cit., p. 16.

de l'Académie»[71]. De la Hire succède à Roberval (1602-1675) au collège de France[72], et prend «beaucoup de soin à ramasser tous les manuscrits des Mathematiciens de notre Academie ce qui avoit esté dispersé d'un costé et d'autre après leur mort»[73]. Dont les archives de Roberval écrit-il à Huygens en septembre 1686. Les méthodes de Florimond de Beaune ont donc pu inspirer les enseignements De la Hire à l'Académie d'Architecture. Desargues est le seul auteur cité dans le traité de coupe des pierres de La Hire, très brièvement, pour la dénomination d'un plan d'inclinaison[74]: «le plan de route».

Une inspiration et une confirmation de l'intérêt que la «*Méthode Générale*» pouvait apporter à la formation des architectes et ingénieurs de l'Académie.
Philippe de la Hire a compris que l'application d'une méthode universelle comme la «*Méthode Générale*» permettrait aux élèves de rationaliser les épures et la coupe matérielle des pierres de telle façon que l'éviction de l'appareilleur soit envisageable sans compromettre le chantier.

Nos recherches sur l'hôtel de ville d'Arles vont dans ce sens, et nous ont permis de démontrer que l'accès aux angles de coupe des pierres de la voûte projetée par Hardouin-Mansart nécessite la connaissance de la *Méthode Générale* de l'angle solide. Le chantier de l'hôtel de ville d'Arles commence en 1673, date du retour du manuscrit de De Beaune à Paris. Dans le cas de Hardouin-Mansart, il faut envisager une connaissance acquise par les travaux de Desargues et Bosse.

Le traité de la «Couppe des Pierres»

Il existe plusieurs copies du traité de la coupe des pierres de Philippe de la Hire.

[71] *Ibidem.*

[72] Il gagne le concours en 1676, voir Auger L., *Un savant méconnu: Gilles Personne de Roberval, 1602-1675*… Paris, Blanchard, 1962, mais ne prête serment qu'en 1683, voir Archives du Collège de France, dossier "*La Hire*".

[73] *Lettre de Ph. De la Hire à Christiaan Huygens du 8 septembre 1686,* dans *Œuvres complètes de Christiaan Huygens*, publiées par la Société hollandaise des sciences, Harlem, 1901, vol. IX, 1901, p. 91.

[74] La Hire Philippe de, *La Couppe*, cit., t. I, fol. 26*v.*

L'exemplaire de l'institut par exemple est une recomposition effectuée par Augustin de la Hire seize ans après la mort de son père à partir des papiers conservés par la famille. Daté au commencement et à la fin du traité, il fut composé entre 1688 et le 6 septembre 1690. Augustin de la Hire décrit le manuscrit comme inspiré des cours de coupe des pierres que donnait Philippe de la Hire à l'académie pour les années 1688 à 1690.
Les autres exemplaires connus, notamment ceux des bibliothèques municipales de Rennes et de Langres sont plus ou moins similaires à celui de l'institut, en revanche celui de la Bibliothèque des Ponts et Chaussés est diffèrent, composé en deux tomes.
Le premier tome est un manuscrit original soigneusement préparé par Philippe de la Hire pour l'impression. Le second tome composé entre 1702 et 1704 est le recueil du deuxième cours de coupe des pierres donné à l'Académie royale d'Architecture.

Cet exemplaire est le plus intéressant, le manuscrit bien ordonné suivant un sommaire en début d'ouvrage, est composé de plusieurs écritures, d'une grande quantité de planches réalisées pour le cours et de commentaires d'élèves.

L'ingénieur du roi "De Chermont"[75] semble être chargé de la mise au propre de ce second traité, certaines pièces de traits du cours sont co-signées «La hyre Chermont».

Le cours de coupe des pierre à l'Académie

Le cours de coupe des pierres à l'Académie d'Architecture au Louvre peut être décrit à l'aide de plusieurs sources, par les manuscrits du cours, les Procès-verbaux de l'Académie, les témoignages d'élèves etc…: les cours publics à l'Académie se tenaient le lundi et le jeudi[76]. Philippe de la Hire présentait le sujet aux élèves par le tracé, graphique, comme en témoigne ce commentaire écrit sur une épure: «du 2e cours de M de la Hire, mais

[75] La famille De Chermont a livré plusieurs générations d'ingénieurs militaires: voir Blanchard A., *Dictionnaire des ingénieurs militaires*, 1691-1791 (ed. Montpellier, 1981). Merci à Guy Picolet pour son aide dans la recherche des ingénieurs du cours de la coupe des pierres.

[76] D'Aviler, A.-C., *Dictionnaire d´architecture civile et hydraulique, et des arts,* Paris, 1755, p. 3.

dont on n'a pu avoir l'explication»[77], il donne aussi des explications, certainement assez peu pour laisser l'espace à la découverte comme en témoigne Frezier: «le trait suivant tiré d'une des leçons de feu M. de la Hire à l'Académie d'Architecture que j'ai énoncé différemment, précédé et augmenté des raisons qu'il laissoit à trouver à ses auditeurs...»[78], ou encore, «comme il laissoit à ses Auditeurs le soin d'en trouver les Démonstrations ...»[79].
Pendant le cours, de la Hire présente un sujet par une épure comme en témoigne les plans parafé LH présents dans le second volume, les élèves le copient à main levée. Il donne les explications nécessaires pour permettre aux élèves de comprendre le sujet, mais leur laisse le choix de la manière de procéder. Pour ce faire, les élèves ont appris la *Méthode Générale* de l'angle solide, mais aussi une méthode pour le tracé des beveaux[80], pour le tracé des arcs, des ovales, des panneaux. Les élèves disposent d'une «caisse à outils» composée de «méthodes» permettant la rationalisation de la géométrie et des procédés de fabrication.

Son cours devait certainement être centré autour de la description des sections coniques en correspondance du volume choisi, la description des propriétés coniques de chaque joint, chaque arête de pénétration est minutieusement décrite dans le texte de Chermont.
Pour de la Hire, le texte peut se passer du tracé: la description des étapes est d'une telle précision que le lecteur peut dessiner l'intégralité de l'épure à main levée sans avoir recourt au tâtonnement habituel des instruments de tracé (règle, compas, ...) rencontré dans les traités de coupe des pierres antérieurs.
Pour parvenir à ce résultat qu'aucun n'avait atteint avant lui, Philippe de la Hire exige de ses élèves des textes très détaillés au vocabulaire très précis. Pour contenir ses explications dans un texte de taille raisonnable, une liste de symboles est utilisée comme en témoigne la photo **[fig. 2]**:

[77] La Hire Philippe de, *La Couppe*, cit., t. II, fol. 174.

[78] Voir Frézier A.-F., *La Théorie et la Pratique de la Coupe des Pierres et des Bois, pour la Construction des Voutes…*, trois volumes, Strasbourg, 1737-1739, t. II, p. 402.

[79] *Ivi*, t. II, p. 2.

[80] A l'instar de la fausse équerre, le beveau matérialise dans une contreforme l'angle à tailler dans la pierre, que les branches de cet angle soient courbes ou droites. Autre appellation: biveau.

> Ces points, on décrira des ellypses 6,14; 7, 13; 8, 12; 9, 11; semblables et [égales[81]] a l'[ellipse] DP. De la [section] oblique, ces [ellipses] seront les profils d'autant de [sections] obliques [parallèles] à la première DK

Les compétences géométriques de la Hire n'ont rien d'étonnant: comprendre et décrire la découpe d'un corps géométrique par des tracés est bien entendu à sa portée. Le dessin complet de l'épure et le texte parfaitement compréhensible autorisent l'élève à couper le trait. Après le dessin, il appartient aux élèves de trouver le cheminement pour obtenir angles et panneaux qui permettent de couper la pierre.
L'organisation de petits concours[82] au sein de l'académie décrit les étapes en détails:

> Du 23e Avril 1703[83].
> La Compagnie ayant cy devant proposé aux étudians de l'Académie de faire le trait géométrique d'une lunette rampante qui rachette une voûte en descente droite, ils ont travaillé pendant plusieurs jours d'assemblée dans un cabinet de l'Académie, remettant chaque jour leurs desseins entre les mains de M. de La Hire, et aujourd'huy trois de ces étudians ont présenté ces desseins finis, que la Compagnie a examinés, et ensuite, après avoir parafé les mesmes desseins et épures, on les leur a remis entre les mains pour les exécuter et couper en pierre.

[81] Le signe égal utilisé par de la Hire en 1688, est 2|2 alors que Descartes a introduit le signe égal dans la Géométrie en 1637 en notant ꝏ, c'est-à-dire d'après le latin ae, début de aequalis. Cependant 2|2 est remplacé par = dans le deuxième tome de la coupe des pierres rédigé entre 1702 et 1704.

[82] - *P.v. Acad. Arch.*, t. III, p. 143. Du 16e janvier 1702. «Monsieur le surintendant, suivant la dernière délibération, a marqué qu'il souhaittoit qu'on proposast aux deux prétendans des prix, dont les desseins ont esté examinés, de faire un plan d'église de vingt thoises au carré.Et ensuite Monsieur le surintendant, pour encourager de plus en plus les jeunes étudians de l'Académie à continuer leurs études, a marqué encore qu'il feroit délivrer des prix tous les trois mois à ceux qui marqueroient avoir plus de génie et d'assiduité aux leçons publiques».
- *P.v. Acad. Arch.*, t. III, p. 169. Du 26e février 1703. «Monsr de La Hire a fait souvenir ensuite à la Compagnie que, Monsieur le Surintendant souhaitant qu'on distribuast des petits prix aux étudians de l'Académie, on devoit leur proposer différens sujets qu'on jugeroit à propos pour les exercer».

[83] *P.v. Acad. Arch.*, t. III, p. 173.

Les élèves du cours sont employés dans les armées du Roi comme ingénieur ordinaire entre 1704 et 1707, de la Hire fut leur instructeur et son influence va certainement au-delà de la seule coupe des pierres dans leur formation. Ces élèves, formés avec un esprit d'initiative, sont capables d'innovation et de création comme le cours en témoigne **[fig. 3]**.

Conclusion

Les ingénieurs-architectes participants[84] aux cours des Académies Royales d'architecture et des sciences sont les premiers à être officialisés sous l'Ancien Régime: un arrêt royal de 1716[85], première ébauche d'un statut de corps, détermine les règles applicables aux ingénieurs ordinaires des Ponts-et-Chaussées, que l'on désigne dès cette date par «Corps des Ponts-et-Chaussées».

Une organisation de «fonctionnaires», dotée d'une position stable garantie par l'État, gouvernée par des règles, dont les membres sont recrutés selon des modalités connues et qui développent un fort sentiment d'identité collective. Un corps que l'Etat oppose à l'ordre professionnel et social fondé sur les communautés de métiers.
En rationalisant les nombreux tracés de la coupe des pierres utilisé par les maçons par des Méthodes Générales, les ingénieurs et architectes du Roi domineront le chantier. La formation par la science s'oppose à l'expérience dans tous les domaines face aux entrepreneurs. Porté par la cohérence du corps des Ponts et chaussés, le projet Delormien formé par Desargues et La Hire voit le jour.

Les tailleurs de pierres ont toujours vu dans l'ancienne méthode un patrimoine géométrique sur lequel repose leur légitimité, l'intégration des méthodes de Desargues et Philippe De la Hire dans les traités de la coupe des pierres du XVIII^{e} siècle restera limitée. En 1728, Jean Baptiste de la Rue donne une seule représentation de la *Méthode Générale* de La Hire

[84] De Chermont, D'Astier et Cercelier, qui ont suivit les cours de La Hire sont cités dans la lettre de De Chermont à La Hire, ils sont tous ingénieurs militaires en temps de guerre à Turin, Aoste et Chivas (Chivasso): t. II, fol. 160.

[85] Arrêt royal du 1 février 1716, Archives Nationales E883, fol. 19-20.

dans tout son traité[86]. En revanche, Amédée-François Frézier (1682-1773) en décrit, en 1737[87], les tracés dans un chapitre appelé “Goniographie ou description des angles”.

Ces méthodes sont finalement reconnues à leur juste valeur par les charpentiers, qui apprennent à les maîtriser et en feront “le trait à la sauterelle”. L’ironie de l’histoire veut que les charpentiers défendent le «trait à la sauterelle» dans une lutte passionnée contre l’obligation faite à tous les métiers d’utiliser uniquement la Géométrie Descriptive après la Révolution française.
La première représentation du «trait à la sauterelle» dans un traité de charpente date de 1748, l’auteur Georg Peter Schillinger[88] l’appellera *frantzösische façon* ou le trait à la Française.

[86] De La Rue J.-B., *Traité de la Coupe des Pierres*, Paris, 1728.

[87] Frézier A.F., *La Théorie et la Pratique*, cit., t. I, 1737, pp. 368-388.

[88] Schillinger G.P., *Architectura civilis*, Nürnberg, Homanns Erben, 1745-1748.

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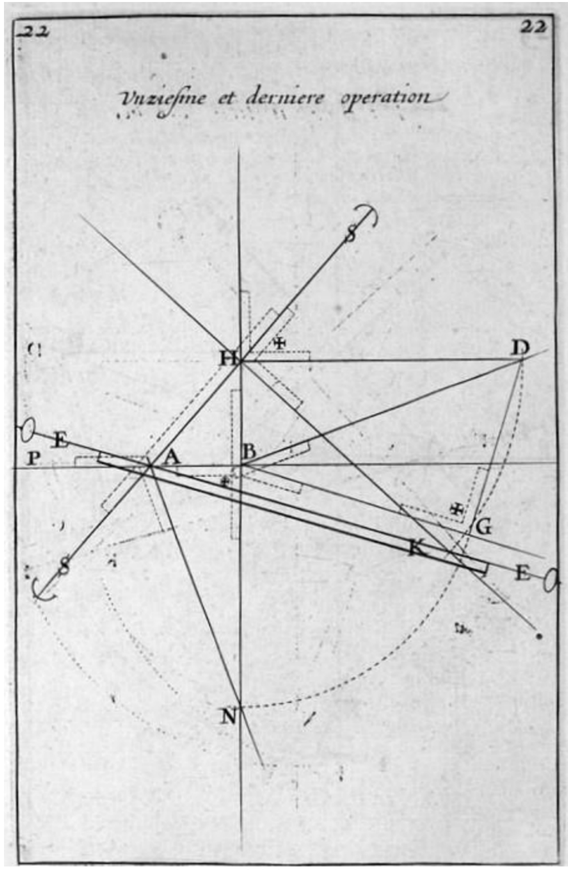

Fig. 1. “Préparation Générale” par Abraham Bosse, 1644.

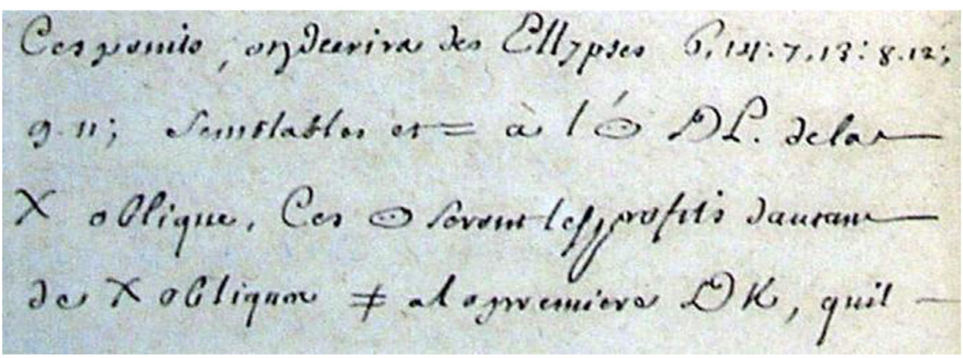
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Fig. 2. “La couppe des pierres”, second cours, feuillet 57, détail du texte. Écriture de De la Hire.

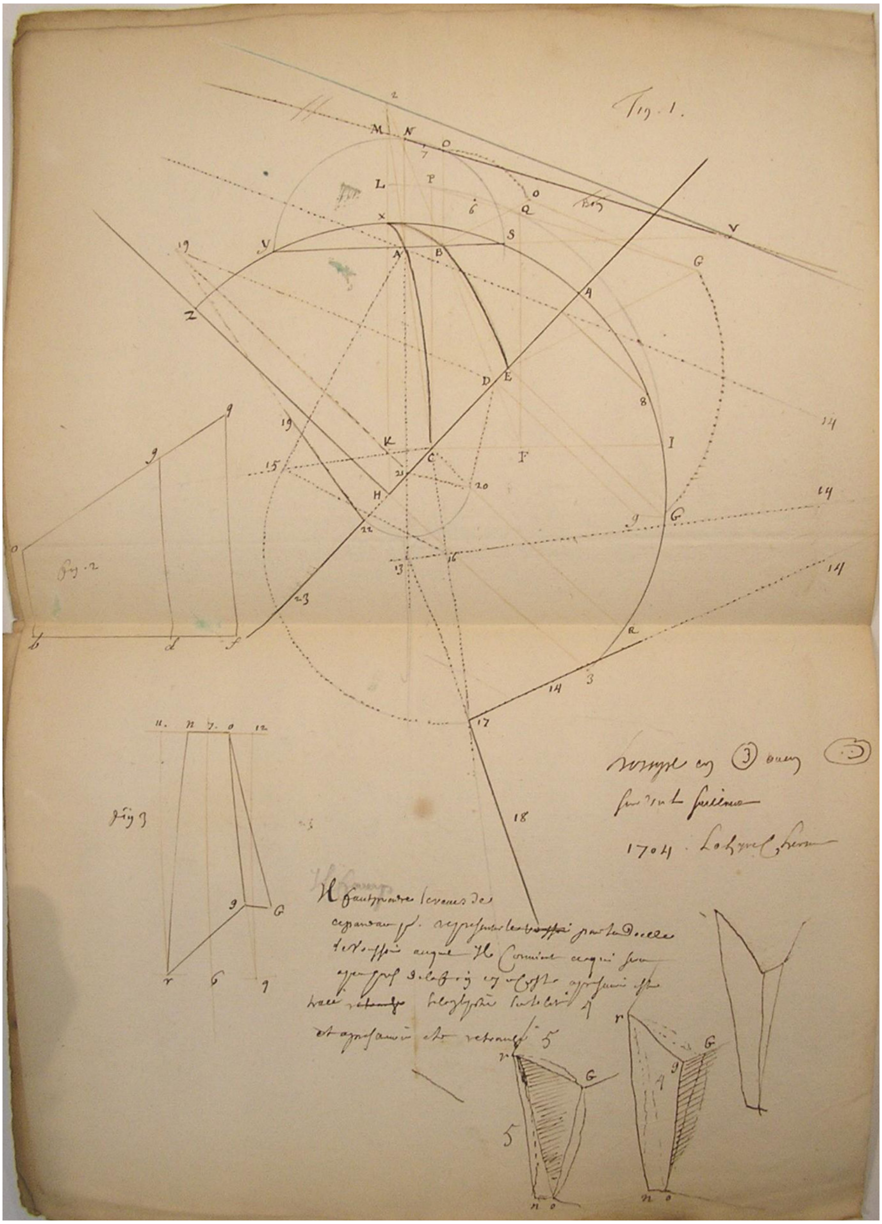

Fig. 3. “La couppe des pierres”, second cours, feuillet 110. L'épure et le texte sont de la main de De Chermont, les étapes de la coupe de la pierre sont dessinées, nous remarquons aussi le paraphe «La hyre Chermont» à droite de la date, 1704. L'épure est la même que celle du feuillet 84 de la main de La Hire.

Denis Zastavni

Robert Maillart's essential contribution to structural design with concrete. Morphogenesis for architectural structures

Introduction

Robert Maillart's works have become familiar thanks to publications by Sigfried Giedion (1941), Max Bill (1949) and Professor David Billington (1973, 1974, 1979, 1980, 1997) (Zastavni, 2012b). Features of their design and the history of their construction are well known. However, a characterisation of Maillart's design methods and other methodological aspects has yet to be written. This contribution looks in particular at the methods used by Robert Maillart in which graphical tools or ways of thinking graphically – in interaction with analytical aspects – play a decisive role.

Given the complexity of Maillart's methods, only certain elements of his work will be presented here. However, they will offer a synthesis of his viewpoints concerning the structural issue.

Robert Maillart

Maillart was a Swiss engineer who worked on bridges and concrete structures from around the turn of the twentieth century until 1940. However, some of his earliest works were for private companies in around 1894.
He worked until his death in 1940 at the age of 68 **[fig. 1]**.

Normalisation of calculations dates from between 1900 and 1905, depending on the country concerned (Simonnet, 2005). At that time, a considerable amount had yet to be invented in relation to concrete bridge structures and defining the best forms for a concrete structure. The same remains true today.

Maillart worked on more than 300 structures – 50 of which are bridges – between the end of the 19th century and 1940 (Rigassi, Billington 1988; Bill, 1955). Initially, Maillart became known for new structural forms using reinforced concrete. At a time when concrete was associated with massive material producing heavy-looking structures, Maillart executed structures with a lightness that had never been seen before. He skilfully produced advanced structural executions that were reliable and highly durable.

This means that it was a matter of entirely rational propositions for rational structural schemes.

Two of his first and best-known inventions date from 1901 and 1905. They are the concrete box girder on the Zuoz Bridge – which he named "Systeme Maillart" – which was patented in 1902, and the characteristic concrete three-hinged arch on the Tavanasa Bridge in 1905. He was 33 years old at the time **[fig. 2]**.

There were subsequent developments to Maillart's three hinged-arch design following the famous Salginatobel Bridge, the next structure of this type, built in 1929: the Vessy Bridge of 1936 which also has his characteristic X-shape columns **[fig. 3]** and his final bridge, the Lachen Bridge, in 1940 are representative of these changes **[fig. 4]**.

In around 1924, Maillart – then aged fifty-two – also invented a characteristic form of concrete arch bridge stiffened by the deck. This followed a principle suggested by Wilhelm Ritter. The principle combines a funicular arch for bearing the uniformly distributed loads and a stiffening member – as a bent girder – which is performed here by the deck, for all other load configurations **[fig. 5]**.

The Schwandbach Bridge of 1933 is his most famous bridge of this kind **[fig. 23]**.
Maillart also worked on continuous beam bridges. These structures – as designed by Maillart – can be seen as borderline examples of a stiffened arch bridge since the arch and the stiffening member have become a unique beam element.

Design methods for three-hinged bridges and the Salginatobel Bridge

Maillart's methods have been explored in depth by Professor David Billington from a chronological and analytical point of view. However very little has been written from a graphical and geometrical perspective.

A series of Graphic Statics drawings can be found in the Maillart archives at the ETH in Zurich.
Despite the fact that very little has been written about Maillart's use of Graphic Statics, the hypothesis proposed was that Graphic Statics were a crucial element in his design methods (Zastavni, 2008a).

Considering this topic of Graphic Statics, two of Maillart's structures are particularly representative of his approach to the structural issue:

- the Salginatobel Bridge, built in 1929
- the Chiasso Shed structure, built in 1924.

There will be a particular emphasis on the latter's design latter since it has been key to understanding Maillart's methods.

With regard to Graphic Statics, in Maillart's archives, it is possible to find (Maillart, 1928; Zastavni, 2009):

- a series of drawings for the purposes of designing the structural form – for morphogenesis – to invent the forms
- drawings with the aim of analysing the structure – to validate the forms and dimensions.

It is worth while exploring the aim of these first working drawings of the Salginatobel Bridge (Fivet, Zastavni, 2012a).

After analyses in order to recompose the drawing, the following seems most likely: the sketch of the bridge has been divided into sections to estimate their respective loads. Loads are then used to draw the *load line* which is used to construct a *force polygon*: extreme segments of the *force polygon* are set at an angle of forty-five degrees, providing a pole at their intersection. For each ray of the *force polygon*, a parallel line can be drawn on the main figure to construct a *funicular polygon* acting in traction.
When the *funicular polygon* has been completed, the two extreme segments are extended to produce an intersection through which the *line of action* of the force resulting from the applied loads has to cross.

In order to create equilibrium, this *line of action* of the resultant of the applied forces must intersect with those of the forces applied at the hinges (situated at the supports and the key of the arch). For the hinge located at the key of the arch, the *line of action* must be horizontal: this is because the arch and loadings are both symmetrical. The *line of action* of the reaction force of the hinge at the arch support is then known:

- it must pass through this intersection of the vertical *line of action* of the resultant of the applied forces and the horizontal *line of action* of the force at the key of the arch.
- it must also pass through the centre of the hinge since there are no bending moments in a hinge.

So the *line of action* of the reaction force acting on the support can be drawn passing through the two points and the orientation of this force

is therefore known **[fig. 6]**. Its magnitude can be measured on its corresponding parallel on the Force polygon.
This allows the foundation block to be studied – graphically – considering the orientation of the forces on the mountain's rocks, and considering the area required to limit the stress on the ground **[fig. 7]**.

But more globally, a new synthesis can be made of this method, considering its design purpose and using a *thrust line* also designed to Maillart's sketch drawings.
Here, first of all the rhythm of interspacing for carrying the deck is studied, as is the division of the arch **[fig. 8]**.

The *funicular polygon* is first drawn acting in traction on the *space diagram* to discover the orientation of the force at support hinges, before going back to the *force polygon* by drawing parallels to the *line of action* of these two forces at the support and at the key of the arch. Their intersection gives the pole of the sought *funicular polygon* acting in compression, future *thrust line*. Parallels of the segments joining this pole and the ends of loading forces on the *force polygon*, give the *thrust line* on the *space diagram* drawn as a compressed *funicular polygon*. It can be seen that this is not that far from the final geometry of the bridge **[fig. 9]**.
The next step is to design the successive transversal sections of the arch, with the aim of limiting bending and compression forces, as detailed in the next section of this paper.

In summary, it has been seen here that graphics do really seem to have been used as a tool for establishing the structural form and some of its details.
Of course, there have been algebraic calculations as well to specify the dimension of the forms, calculated in correlation to the drawings. This will be explored below.

Designing sections with concrete boxes from Maillart's work

While working drawings in elevation using Graphic Statics demonstrate the method's ability to devise the average geometry – here it can be called the "*typology*" – the constituent parts of this geometry also have to be designed and dimensioned for resistance. Working drawings found in Maillart's Archives in Zurich show two things: the existence of a significant quantity of work on details and the degree of elaboration of the work of detailing. Considering the Salginatobel Bridge **[fig. 10]**, the drawing of the

section of the deck shows that multiple variants have been considered: three profiles of the route slab, the nature of the parapet in concrete or steel and at least three variants for the details of the lateral limits of the box section.
In parallel, the dimensions of the successive transversal sections of the arch have been adjusted for their centroid to correspond at best with the trajectory of the *thrust line* in the sketch of the bridge in a lateral view. The respective position of the centres of gravity compared to the *thrust line* can be managed by adjusting the upper and lower limits of the arch successive sections.
From this point of view, Graphic Statics have been used recursively to define the trajectory of forces around which the profile of the bridge has been drawn.

Maillart's design of stiffened arch bridges

With Maillart's three-hinged arch bridges, it can be seen how Maillart proceeded to elaborate more complex structural behaviours into specific geometrical arrangements. As shown elsewhere (Billington, 1973), the stiffened arch bridge is the complementary association of a funicular arch supporting permanent loads by compression only, and a rigid deck acting as a girder against live loads **[fig. 11]**. This bridge corresponds to the structural model of a suspended bridge but with an inversion between compression and traction forces considering the funicular member. This was suggested by Wilhelm Ritter when Maillart was still a student (Billington, 1997).

Maillart proceeded by dissociating both mechanisms of compression in an arch and bending in a girder. The stiffening girder is supposed to lean on the centre of the arch and at both ends. Maillart studied the structural behaviour of the arch and the deck separately, paying no attention to the interaction between these elements and with the columns connecting the desk and the supporting arch. This simplification of the problem appeared unusual to Maillart's contemporaries and was interpreted by them as a negligence. They perceived these bridges as arched Vierendeel-like structures with the characteristics of the various members being radically different and therefore needing very complex analysis due to bending. On the one hand, they did not consider the respective stiffness of the constituent members of the structure which influences the way the forces will be distributed between the various members; on the other, they themselves paid no attention to the geometrical parameter linked to the presence of a funicular arch combined with conditions in the foundations that could support horizontal thrust. This means that, against the principal forces the first mechanism to be activated is an arch mechanism instead of a framed girder mechanism.

In order to design such a bridge, Maillart provided the dimensioning of the respective members on the basis of his simplified models. At the same time, to be sure that no undesired interaction occurred between the constituent parts of the structure that would challenge the structural behaviour that he selected for the dimensioning, he made sure that the ratios of proportions of the constituent elements' mechanical properties (stiffness) were radically different, further adding another type of property to this: the "geometrical" stiffness (against axial forces depending on the kind of loading). This "geometrical" stiffness was designed using Graphic Statics to define the suitable geometry. All these elements – a funicular slender arch, the supporting concrete walls with a thickness reduced to a minimum and a deck rigidly combined with concrete parapets to form the stiffening girder members of the bridge in the form of a U-section – are simply combined to form the final design. For instance, in the Valtschielbach Bridge built in 1925, the thickness of the arch varies between 23 and 29cm, the transversal supporting walls are 15cm and the deck develops a stiffness of 1,22*10^7cm^4, i.e. nearly twenty times the stiffness of the arch **[fig. 12]**. This structural method of design by adding separate structural mechanisms has been linked to plastic design in a decisive contribution made by J. Ochsendorf (2005)**.** It enables simplified calculations to be perceived not as a dereliction of duty or an easy way out, but as an illustration of the use of the lower bound theorem of plastic design. Maillart can now be seen as something of a forerunner, since theorems of plasticity would not be fully developed until the middle of the 20th century, after his death, whereas his contemporaries considered him negligent. Therefore, the use of approximations becomes a central part of a consistent design method.

Robert Maillart was 52 years old when he built his first bridge of this type in 1924. It was therefore the work of a mature engineer and he actually demonstrates an elegant way of managing the complexity in designing a structure while keeping the structural behaviour under control. This kind of approach remains universal in the context of the application of the lower bound theorem of plastic design.

The Magazzini Generali in Chiasso

The design of one of Maillart's strangest structures can now be analysed: the Magazzini Generali in Chiasso built in 1924 **[fig. 13]**.

The buildings include a warehouse, an office building and a covered shed which are predominantly of interest to us. The purpose of this structure was to shelter goods arriving by rail on both sides of the construction. The structure has to be wide open to the space in front of it and on both sides.

First, the geometric arrangement and an analysis of the mechanical properties of the sections should be undertaken. Examining a section on the roof structure and having studied its genesis in geometry, this structure appears to be a stiffened funicular structure, not a complex frame. In the upper part the stiffening member is a T-beam while below is an inverted arch (acting) in traction (Zastavni, 2008b).

Geometric rules can be identified **[fig. 14]**:

- the warehouse building was designed first
- the columns of the shed are aligned with those of the building
- the structure is situated between blue lines corresponding to the floors
- the connections of columns and struts – in red – are located at the same level as the connection of the columns to their capitals
- the connection between the columns and the foundation is level with the connection between the columns and capitals underground.

This is the geometrical pattern that serves as a basis for the design drawing for the structure of the shed.

For the design of the shed itself, the following elements are given: the position of the roof, the position of the columns and the interspacing of the vertical member – being twice that of the store's columns. A uniformly distributed load, corresponding to the snow loads that dominate, is used to construct the *load line*. Robert Bow's interval notation are used below (Bow, 1873). With interval notation, the numbers in the main diagram (*space diagram*) represent areas restricted by structural elements and the capital letters represent spaces between the *lines of action* of the applied forces, so that the bars of the structure are labelled by two numbers or by a number and a letter. The corresponding small letters are used on the *force polygon* to label the two marks defining the length of the vectorial force associated with a bar or an axis in the main diagram. The lengths on the *force polygon* define the forces' magnitudes at scale. On the main figure, forces are labelled with capital letters placed directly on either side of their *lines of action*, which are the trajectories of the forces up to the point of contact with the structure. Zalewski and Allen (1998) provide a clear overview of the graphical design procedures using Bow's notation.

Parallels to the roof parts are then drawn.

Ideally only vertical loads are wanted on the columns and attempts will be made to avoid horizontal thrust.
As soon as the pole "*o*" is placed on the *load line*, a funicular polygon can be drawn which is horizontal at the centre by setting the ray starting from *o* horizontally **[fig. 15]**. An intersection is obtained in *8* between this line *8-o* and the parallel line to the roof slope *8-g*. A vertical line is drawn from *8* defining points *2, 4* and *6* that enable to rays *2-o*, *4-o* and *6-8* to be defined corresponding to the remaining segments of the funicular members on the *force polygon*. Using parallel lines, a funicular line is obtained on the *space diagram* in blue.
However, this version is too low to be functional when the central member remains horizontal.

This can be corrected by extending the forces on the *force polygon* corresponding to the funicular line. Starting from the *force polygon* by sliding dot *8* to make it further away from pole *o*, the slope of the central segment of the funicular line becomes opposed to that of other segments, producing a lower point on axes *F-G* and *H-J* on the space diagram. The reason for this is the incline of the roof.
From this point, geometric constraints are used to manage the continuation of the drawing.

In order to define segments of the funicular line in accordance with the constructive constraints (remaining above the thickness of the capitals in the warehouse), an axis line can be drawn between:

- a point situated on axis *FG*, its level is also known from the level of the floor
- a point situated on the roof and on an axis for equilibrium **[fig. 16]**.

On the *force polygon*, the parallel to the axis line is situated starting from the pole. An intersection is obtained with the roof's parallel, defining the corresponding ray.

Again, a vertical line is drawn, to intersect the parallels to the roof slope, defining the rays of the future funicular line. An adapted funicular line is obtained using parallels to the rays on the space diagram.

After proceeding in that way, the geometry of the funicular obtained fits perfectly with the centre of the geometry of the structure (between *D-E* and *K-L*), but is not correct on the two sides for the extreme segments of the funicular line **[fig. 17]**.

Completing the structure will reveal that more adjustments are required to guaranty structural equilibrium. Drawing the two struts placed at the head of the columns, with their corresponding lines as a dashed line 1-A and a blue one *1-P*, shows that the equilibrium is not correct on the *force polygon*: points "*o*" and "*p*" should correspond **[fig. 18]**. This is done **[fig. 19]** by translating the rays corresponding to the funicular line. Then the line "*a-p*" can be drawn on the *force polygon*: it corresponds to the *thrust line* of compression forces along the columns.
When drawn on the main figure by a parallel to "*a-p*", we see that the resulting force *A-P* goes beyond the envelope of the column, and almost outside the foundation. It will be difficult to build because of huge bending moments in the columns, and another type of foundation would have had to have been designed.

There is a way of equilibrating these forces and correcting the trajectory of the resulting vector.
Proceeding graphically **[fig. 20]**, the resulting vector is drawn to remain inside the column's thickness with, as the hypothesis, a minimum of fifteen centimetres from the edges of the columns.
This method consists of arranging the other forces to guarantee equilibrium around this vector AP once it has been adapted to fit the columns. Some adjustments are necessary on the structure: line *1-A* is translated and the orientation of *1-P* is changed. On the *force polygon*, corrections will also be needed to join "*o*" and "*p'*".

The equilibrium can be corrected on the *force polygon* by extending "*a-p*" to the horizontal symmetry axis, giving "*p**". Therefore "*1-p*" must be changed **[fig. 21]**.
Drawing parallels to these new lines on the main figure allows us to obtain the "final geometry", even if it is not plenty equilibrated on the *force polygon*. It all now fits perfectly together and with Maillart's drawing of the structure (with a tolerance between 3 and 6 centimetres): at a scale

of one to fifty (1:50) the correction is a maximum of 1 millimetre on the working drawing.

Using computer program analysis to evaluate this structure shows bending moments in the upper member and there are almost no bent sections in the funicular line and the vertical struts **[fig. 22]**. All the bending moments are encountered in the upper T-beam, which is the stiffening member. Bending moments are also encountered in the columns which also have a T-section. The principle of the stiffened structure has therefore become effective.

If there is a desire to change the geometry to make the bending moments disappear, some points have to be moved by less than 5cm (a little more in the columns up to 15cm): 1 mm (to 3mm in the columns) on a plan at a scale of 1 to 50. This means that it is not realistic to improve the geometry in the drawing and makes all other corrections in the geometry practically meaningless.

To be sure of the exactness of all these drawings, a CAD system with dimensions at full size has been used. The reference geometry comes from dimensions mentioned on Maillart's detailed plans.

The study of Maillart's Chiasso Shed demonstrates the way Graphic Statics can be used as a tool to elaborate and regulate the structural arrangement while interacting with the geometry for maintaining or modifying the equilibrium of force in the structure. Up to this point, it has been shown that Maillart used reference loading cases to elaborate the sketch of the typology of the structure, and recursive drawing and elementary calculations to dimension the successive sections and regulate their behaviour. It has also been shown how Maillart completed structural arrangements with complementary structural mechanisms and associated them, how he sometimes used geometrical patterns to elaborate the bases of the typology and how he managed the equilibriums graphically to makes some structural arrangements in accordance with what he wanted to happen as structural behaviour.

The use of geometry

Maillart built using concrete. Today, reinforced or prestressed concrete is considered an alternative material to steel or timber which are materials intended to sustain bending forces. A specific way of calculating them en-

sues once they are integrated into a structure. Such structures are computed as more or less complex frames and a registry of related typologies accompany their design. From this, there are markers for their structural expressions and detailing of the required finish for specific assemblies to give the structure its expression.
With concrete, a large area of structural behaviour is possible depending on the geometry of the structural element and its composition (type of section and reinforcement). With reinforced concrete, designers started to reproduce structural forms that came from other materials and it took time for really suitable structural arrangements to be invented. Maillart proposed very characteristic typologies for concrete and all proved very well suited to it. Yet could there have been rules – structural behaviours, natures of the stresses etc – that naturally give the concrete its form in the structure?
Maillart gave concrete specific statuses in elements of his structures. The concrete inside hinged arches or funicular arches for stiffened arch bridges is intended to be compressed and a design method based on *thrust lines* was used to design them. Sections remained rectangular and their areas are directly proportional to the force encountered. Elsewhere, some sections were parts of very slender funicular stretched members and were designed with just the required amount of reinforcement steel protected by a concrete skin. Again, a funicular line was used and designed with funicular polygons in Graphic Statics. For some parts of his structures, elements were subject to bending and therefore needed to develop stiffness against bending forces. Sections such as boxes, simple or multiple Tees and U-shapes were considered in this case. Calculation methods were then based on bending calculations or on off-centring the *thrust line* inside arches. When Maillart's structures were sketched, he relied on funicular geometries that came from drawings with Graphic Statics or from geometries known for being associated with specific structural capacities such as parabolic lines for arch actions, for instance in the capitals of mushroom-slabs or connections from the continuous girder to their supports. Hyperbolic lines were also considered for mushroom-slabs against punching forces. All these geometrical figures match specific structural behaviours but more surprisingly they were frequently reinterpreted in the final geometry. Some figures corresponding to the exact mathematical rule expressing the evolution of forces where translated (for instance in hyperbolic capitals of the mushroom slabs: Zastavni, 2012) or inclined (parabolas used obliquely or horizontally to sketch three-hinged arches).

This shows some features of the graphical counterpart of Maillart's design work: his bridges were drawn and architecturalised through geometry together with the analytical or graphical calculations. During the first steps of the design, rounded dimensions were given to sections and forms were ruled according to specific geometries as described above. This attention given by Maillart to the form and to geometry is also a major component of his approach, vital for understanding his design and the methods that produced its features.

Structural choices related to the context of the construction

Maillart's work presents various types of structural systems. Some of these systems are typical for structural applications such as mushroom-slabs. However, when considering bridges, the question is whether the various typologies are interchangeable. Could a three-hinged arch be replaced by a stiffened arch bridge or vice versa? What really distinguishes arch bridges from continuous girder bridges? What were the reasons for the choice that was made?

The following parameters seem to govern or intervene in the choices: the nature and importance of live loads, the nature of the site and particularly the characteristics of the ground in supporting the structure to be built and the geometrical features of the environment.

In the second half of his career, Maillart had already developed five different structural types for bridges in concrete: the stiffened arch bridge, the three-hinged arch bridge, the arch with a strongly off-centre *thrust line*, the continuous girder bridge and, later, cantilever bridges. When he was free to choose the features of the bridge to be built, he first evaluated the bearing capacities of the ground to make his decision as to the most appropriate structural type. Hinged arch bridges were suitable for ground with poor loading capacities. At the start of his career, when concrete bridges were still in part a new venture that had not been fully mastered, the precaution of hinging bridges was the rule. Choice became possible later. Indeed, a stiffened arch bridge can only be functional on very good supporting ground. If that was not the case, the transmission of the compression forces through the funicular arch was deficient, leading to displacements of the supports that have an impact on the structural behaviour, causing the "geometrical stiffness" to vanish and bending forces to appear in all the members like Vierendeel-type structures. With the exception of the circumstance of having good supporting ground, Maillart had a choice of structures that are (relatively) insensitive to horizontal movements, such as three-hinged arches or structures developing no horizontal thrust at their support, such as continuous bridges, arches with an off-centred thrust-line – which means arches sustaining bending forces in a hyperstatic way – or cantilever bridges. The latter did not feature that much in Maillart's career (there are some examples consisting of a variation of the continuous girder bridge). The arch bridge is most appropriate for longer spans, while girder bridges were suitable for lower bridges and for managing intersecting traffic directions. Arch bridges with off-centre *thrust lines* were considered variants of the three-hinged arch bridge and this solution was viewed positively by Maillart even if the people commissioning him preferred hinged-arch bridges.

Stiffened arch bridges were generally higher and, based on the demands made on Maillart, it is interesting to observe the hesitations he had in 1928 in the competition for the bridge to replace the original three-hinged arch Tavanasa bridge (1905) (Billington, 1979). On the one hand a higher bridge enabled a lateral lane to follow the direction of the river to be spanned and on the other a lower three-hinged bridge would be integrated better in the morphology of the site.

When it comes to choosing a structural typology, it is actually about structural behaviour.

The influence of the nature of live load also influenced Maillart's choices. It is to be observed that the heaviest loaded bridges were built as stiffened arch bridges or continuous girder bridges. In these bridges, there was the size of the dead load as well as the supporting gravel layers of the railway and the importance of the loading rail itself, with no vertical sag of the structure acceptable. Considering the magnitude of live loads, their influence is limited proportionally by the magnitude of pre-existing permanent (dead) loads. It all depends on the type of loading: some of the stiffened arch bridges support heavy trains and railways and others only support light traffic loads. When Maillart uses a funicular geometry as a reference, it should be asked to which loads it corresponds and this choice involves the importance of forces found beneath the other loading cases. This also determines which kind of loading has to be chosen as permanent and this influences creep. Indeed, since concrete creep only happens beneath permanent or quasi-permanent loads, an appropriate choice of the reference load for the funicular geometry means different deformations due to bending do not occur to modify this geometry.

The following strategies can be observed in Maillart's work: for most of his three-hinged arches, only dead loads are used as the reference loading case. He designs the locus of the centres of gravity of the successive sections of the bridge to correspond, or best to approach, the *thrust line* under the reference loading case.
When it comes to live loads, the bending forces are evaluated via a *thrust line* under loadings considering its deviation compared to the locus of the centre of gravity.
Most of the time, these bridges are unloaded so that very little bending is associated with the reference loading case, implying a very good hold with concrete remaining compressed over time.
When Maillart designed stiffened arch bridges with a light traffic load, he followed the same logic: the geometry of the funicular arch corresponds to dead loads only and the stiffening member (deck) is activated only against live loads that do not scale the reference loading case proportionately. Again, the arch holds well over time. Differences appear for some of the heavily loaded stiffened arch bridges and continuous girder bridges. For these, Maillart chose dead loads and half of live loads as the reference loading case. This means that bending is reduced for the mean loading and diverges as positive or negative bending from half of the extension of the whole bending caused by the full live load. This last option saved the amount of reinforcement steel associated with bending in the sections.

In summary, various typologies exist in Maillart's work and a correlation can be emphasised between them and the data coming from the physical context on one hand and the use of the structure on the other. Exploring the parameters of those external and internal constraints shows dedicated strategies, sometimes to maximise the hold of the concrete over time and sometimes to optimise the amount of reinforcement steel inside the concrete sections.

A contemporary perspective of Robert Maillart's approaches

Compared with such an expressive and rational way of approaching the design of structures, the approach followed to design structures today should be reconsidered.

Nowadays, the approach to the form and to the structure is frequently managed separately. The most common way of considering the structure is to proceed by way of structural analyses on the basis of a geometrical proposition. If the form is proposed on the basis of environmental, spatial or aesthetic considerations only, it means that the form is not questioned on the basis of structural considerations, except maybe as a consequence of the results of the calculations. More global considerations should therefore be taken, such as integrating rationality in the design and considering form as an unavoidable parameter to achieve objectives such as durability, reliability or sustainability. Unfortunately, structural design today is all about resolving a series of problems involving mechanics.

Frequently the methods used in engineering approaches do not consider the relevance of the form, adapting the forms if structural analysis is used, or the expressiveness of the structure. It is the syndrome of the dichotomy that exists between engineering methods and all the various elements that make up the design of structural arrangements.

Technology allows the situation of a dichotomy to be maintained considering structural design. Except that environmental problems nowadays might perhaps force efficiency to be reconsidered.
In this matter, Maillart's approach to designing structures, which involves thinking globally about the structural behaviour and integrating structural elements into the whole, reveals strategic for designing rationally **[fig. 23]**.

Graphic Statics permitted him to guarantee that vectorial equilibrium was satisfied, so that he achieved mainly efficient axial forces just as is done today with design attempts for structures using struts and ties according to plastic theory. By using graphical methods to determine the form, Maillart created original and elegant structures for his designs. Even though his tools were less sophisticated than those around today, it can be presumed that, even in Maillart's day, it might have been a question of choice rather than necessity.

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Fig. 1. Robert Maillart (1872-1940) – credit: ETH-Bibliothek Zurich, Image Archive.

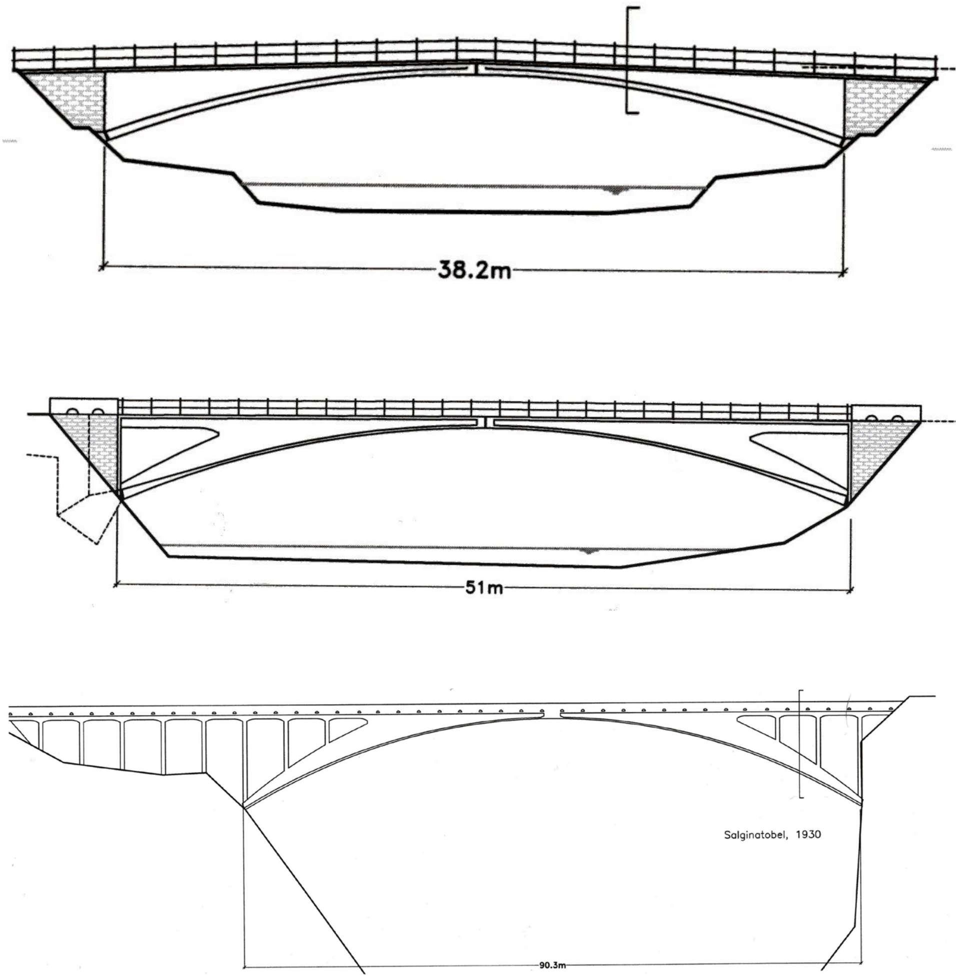

Fig. 2. Robert Maillart’s Zuoz (1901), Tavanasa (1905) and Salginatobel (1929) Bridges – credit: D. Zastavni.

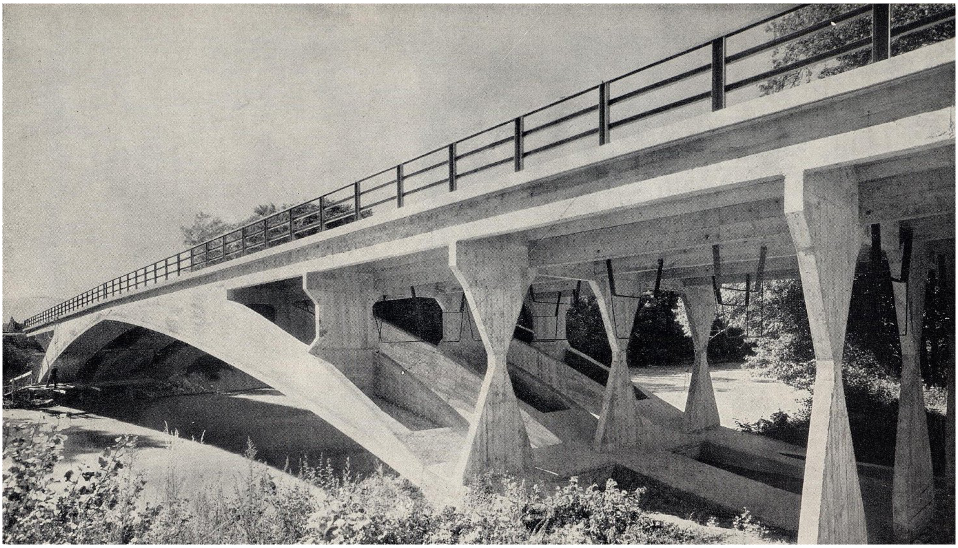

Fig. 3. Maillart’s Vessy Bridge (1936) – credit: ETH-Bibliothek Zurich, Image Archive/ Robert Maillart Archive.

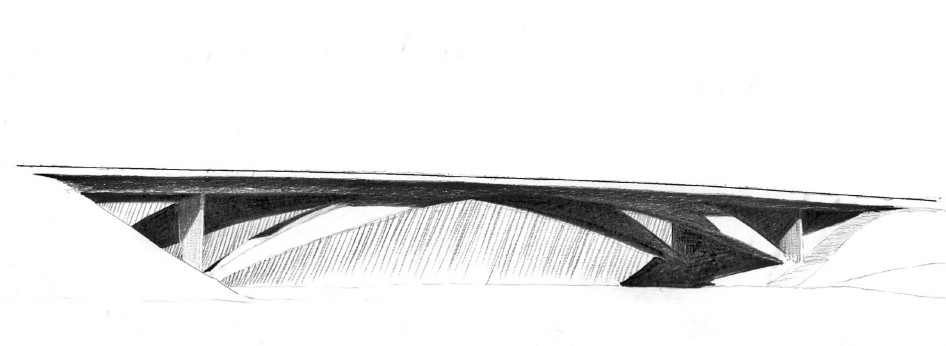

Fig. 4. Maillart’s Lachen Bridge (1940) – credit: ETH-Bibliothek Zurich, Image Archive/ Robert Maillart Archive.

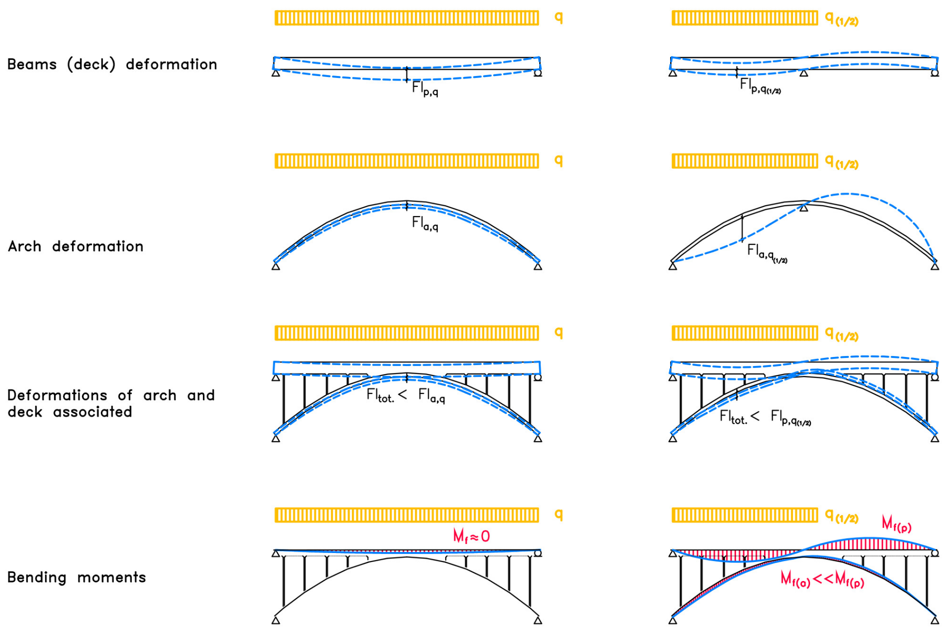

Fig. 5. Principles of Maillart's stiffened arch bridges (credit: DZ).

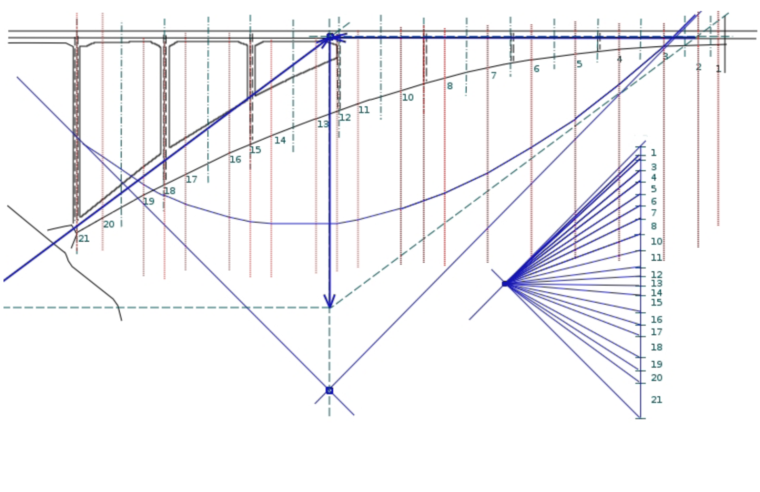

Fig. 6. Working drawing of reaction at supports in Robert Maillart's Salginatobel Bridge (1929) – credit: D. Zastavni.

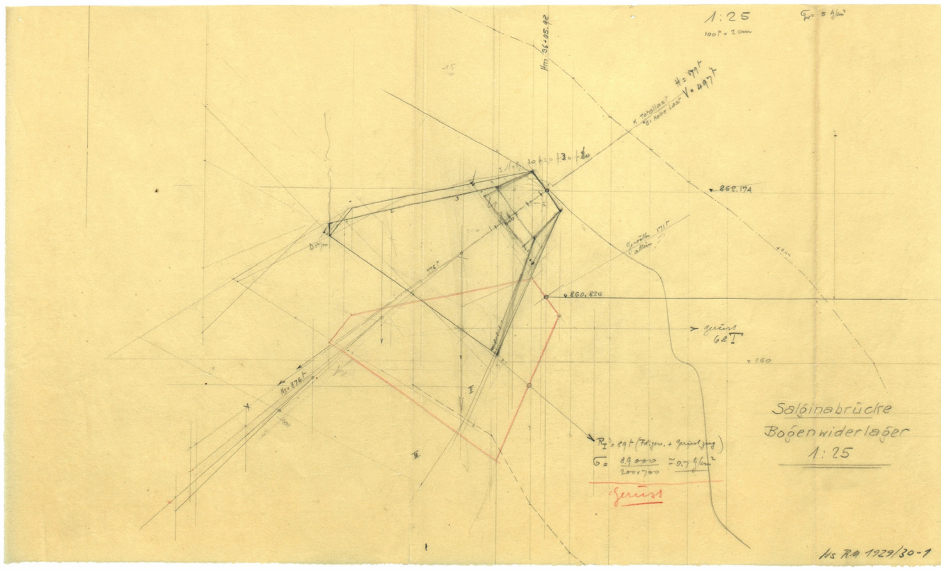

Fig. 7. Working drawing of support in Robert Maillart’s Salginatobel Bridge (1929) – credit: ETH-Bibliothek Zurich, Image Archive/Robert Maillart Archive.

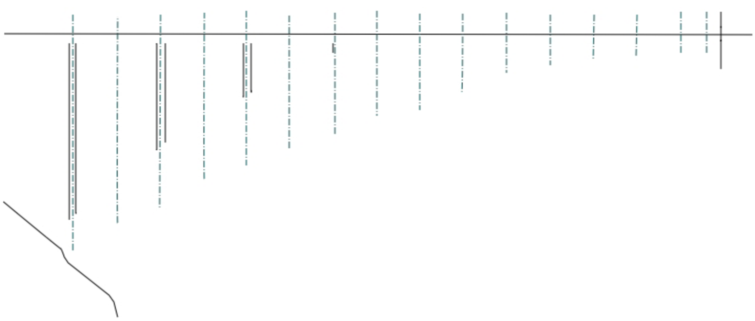

Fig. 8. Working drawing of Maillart's Salginatobel Bridge (1929) for design purposes: loads (credit: DZ).

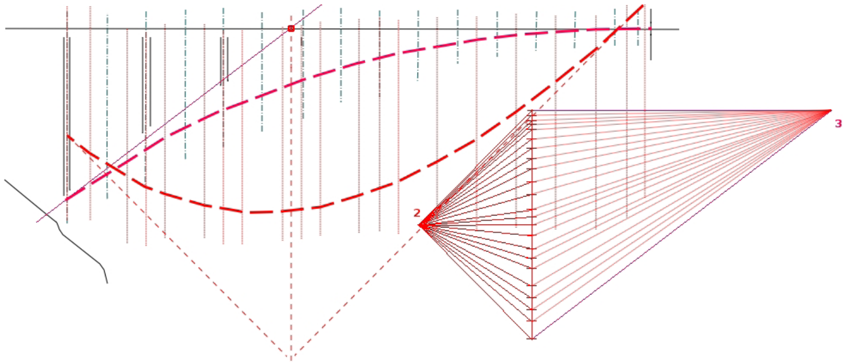

Fig. 9. Working drawing of Maillart's Salginatobel Bridge (1929) for design purposes: thrust line and geometry (credit: DZ).

Fig. 10. Salginatobel Bridge (1929) (credit: DZ).

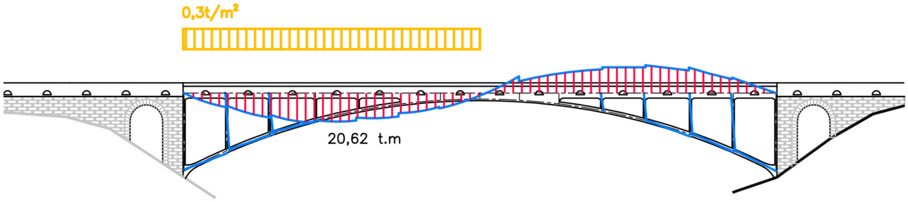

Fig. 11. Valtschielbach Bridge (1925): bending moments under asymmetric live loads (credit: DZ).

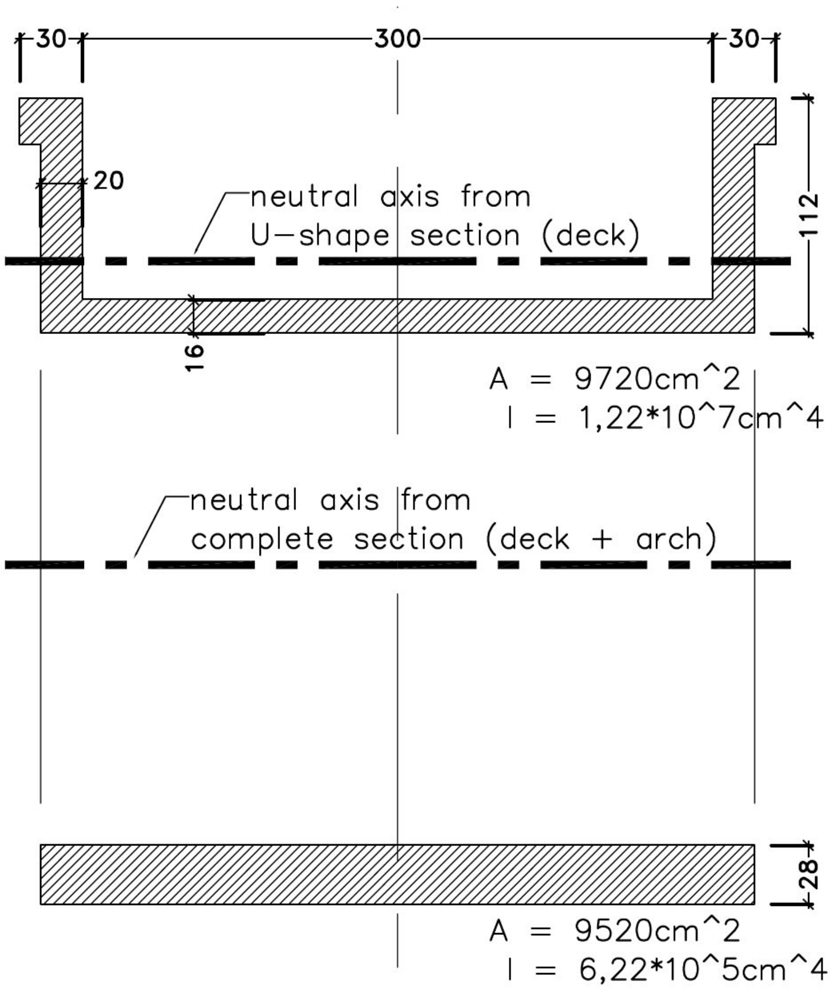

Fig. 12. Valtschielbach Bridge: stiffness ratios (credit: DZ).

Fig. 13. Robert Maillart's Chiasso Shed (1924) – credit: ETH-Bibliothek Zurich, Image Archive/Robert Maillart Archive.

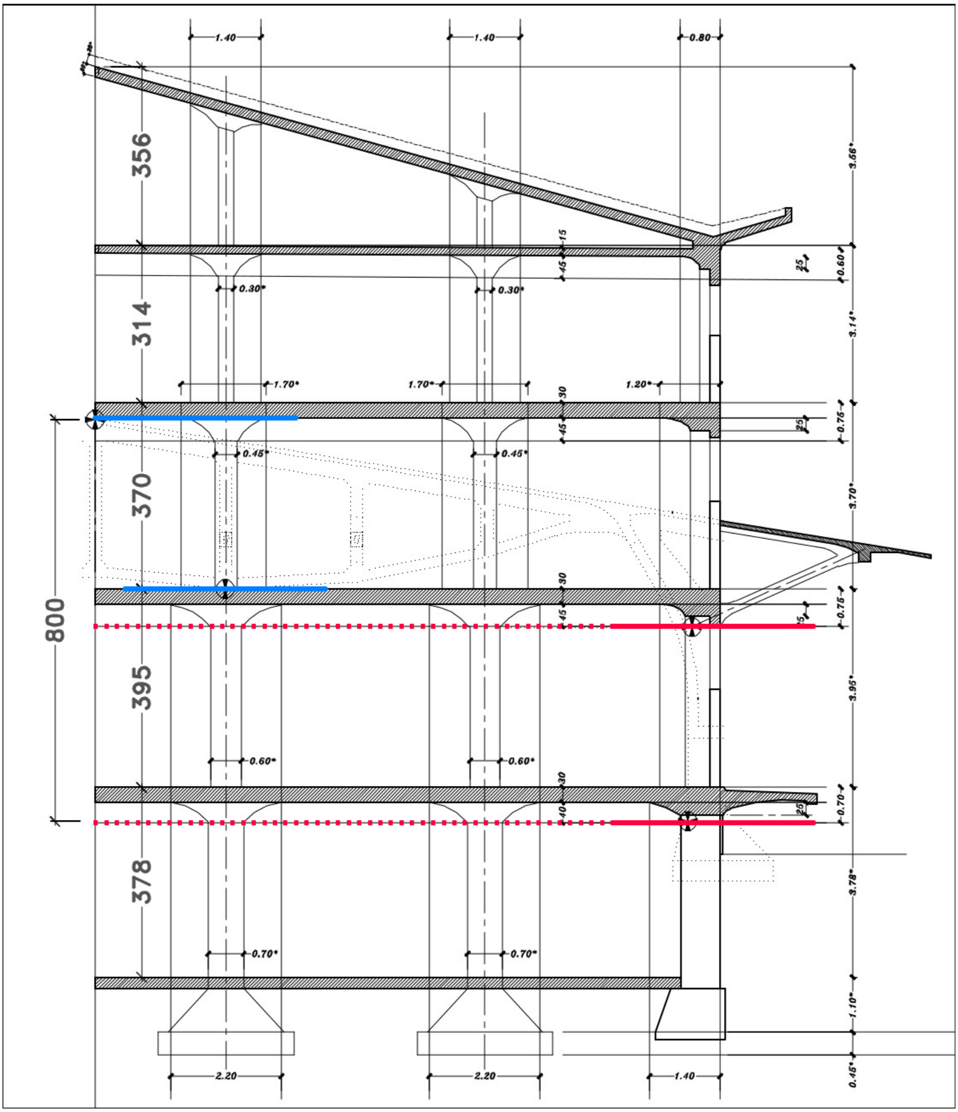

Fig. 14. Shed section superimposed onto the warehouse construction lines.

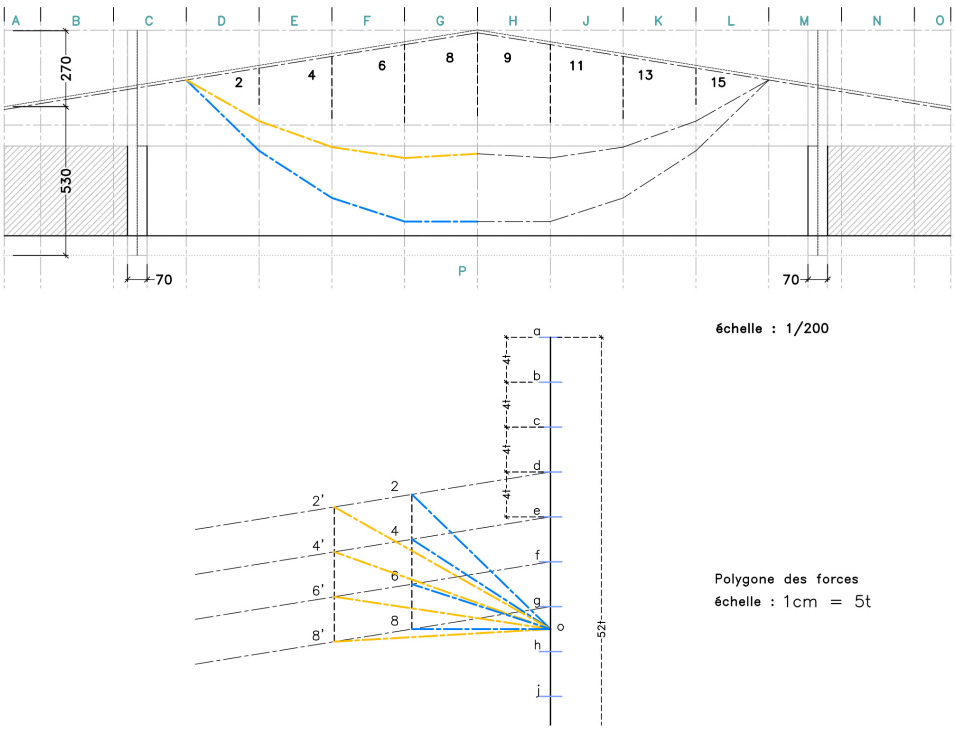

Fig. 15. Working drawing for the Chiasso Shed (step 1): two trial funiculars: the blue one is horizontal at the centre.

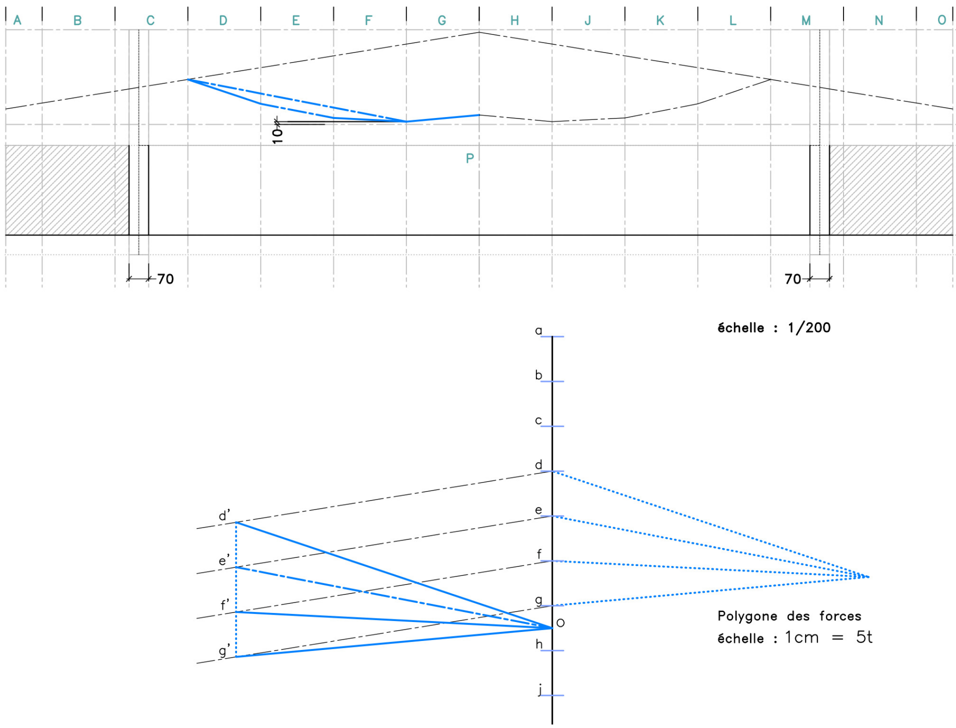

Fig. 16. Working Drawing for the Chiasso Shed (step 2): point o is placed at the mid-point of gh, the orientation of e'o is given by the main figure (between the intersection of the CD axis with the roof's, and a point on the FG axis 10cm above the first floor slab); d', f' and g' are vertically aligned on e'.

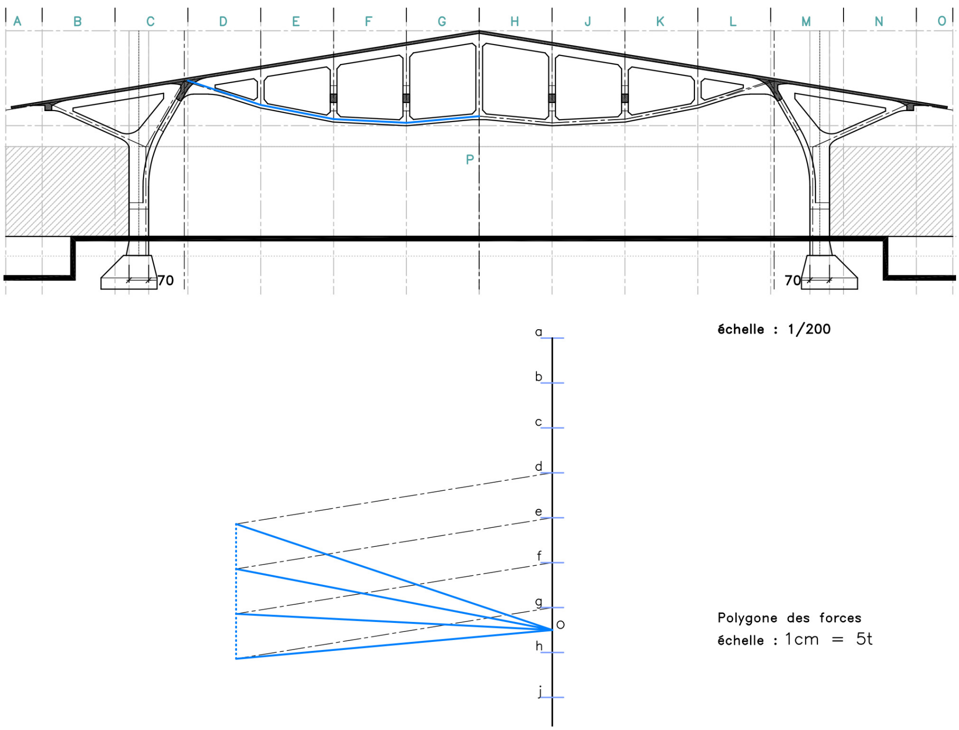

Fig. 17. Working drawing for the Chiasso Shed (step 3): funicular polygon corresponds to the lower chord of the structure's geometry for segments E, F, G; segment D follows other rules.

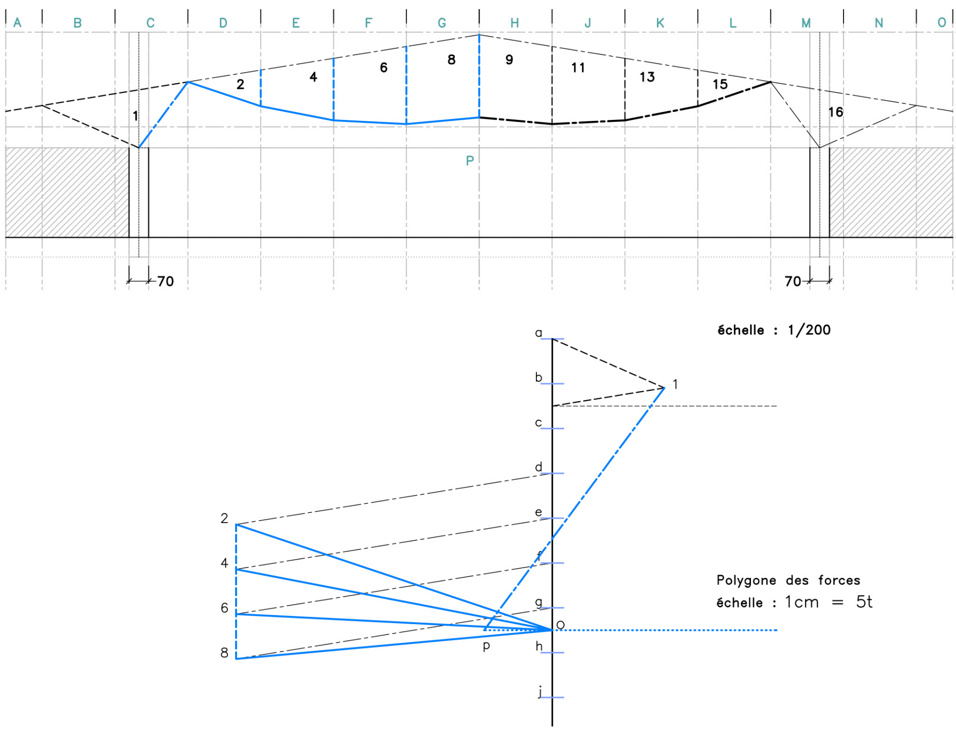

Fig. 18. Working drawing for the Chiasso Shed (step 4): drawing the vertical struts on both figures and constructing the connection with the column; on the force polygon, p is defined as the intersection between the blue axis line and the horizontal line at the mid-point of vector gh. However, o and p should correspond.

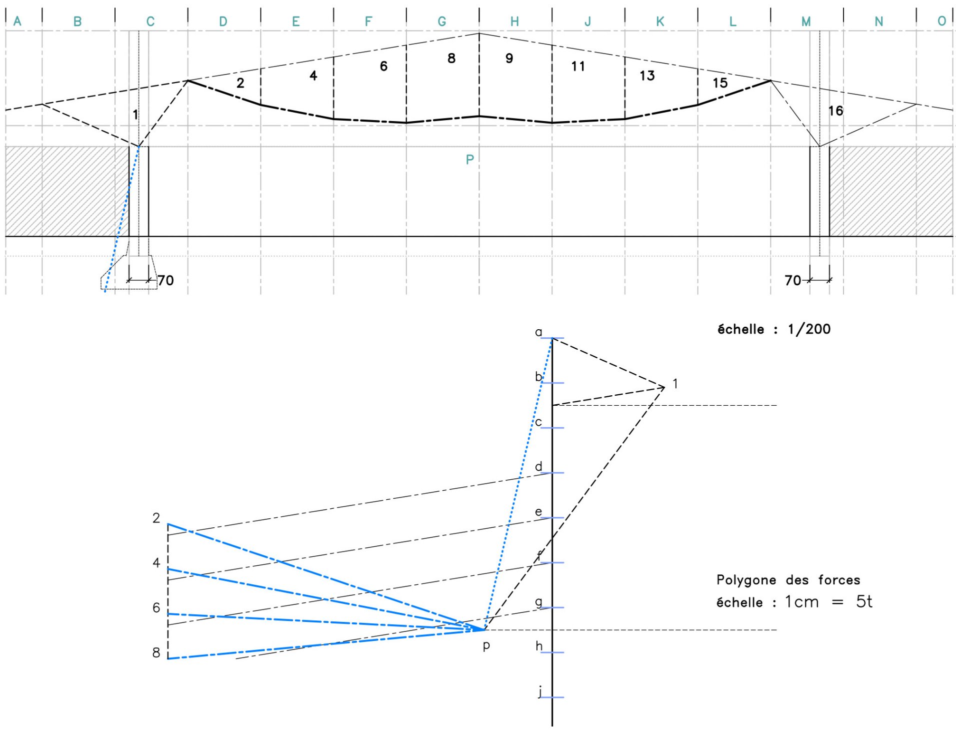

Fig. 19. Working drawing for the Chiasso Shed (step 5): corrected force polygon (pole of the funicular rays is translated on point p); the resultant force in the column is obtained by drawing the dotted line ap.

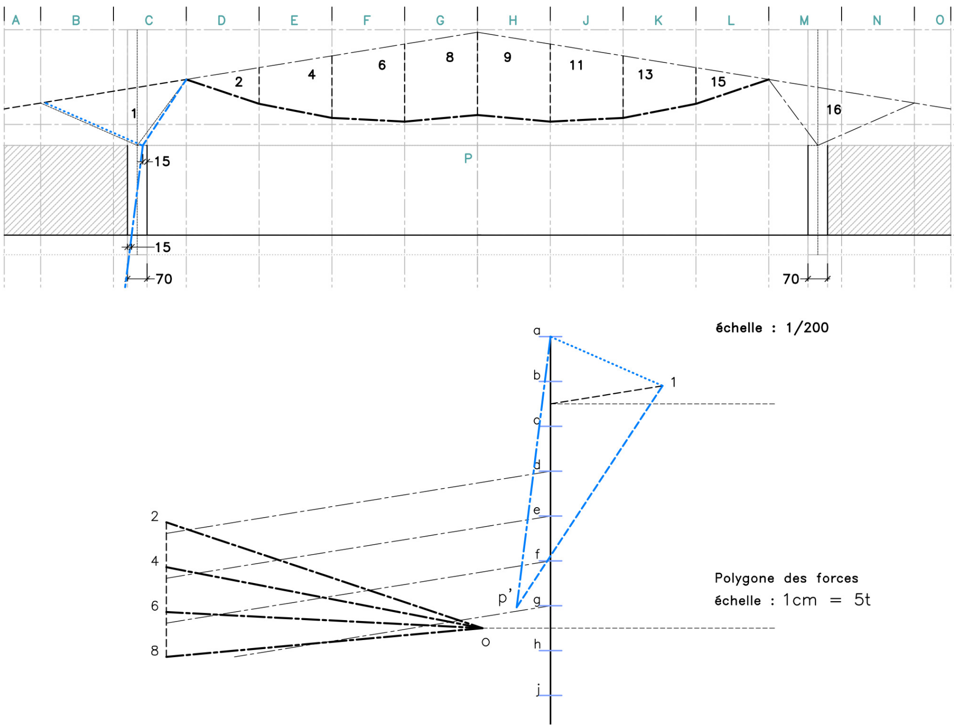

Fig. 20. Working drawing for the Chiasso Shed (step 6): adjusting the resultant force inside the column, and therefore correcting the struts' axes. The force polygon is not yet entirely correct since o must correspond with p'.

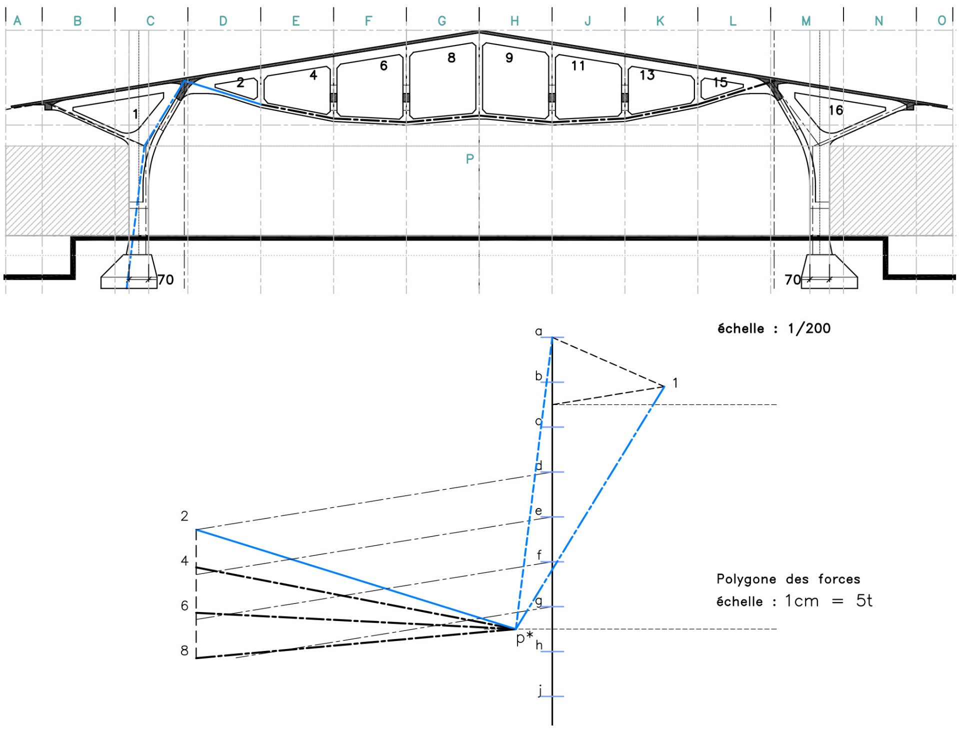

Fig. 21. Working drawing for the Chiasso Shed (step 7): the equilibrium geometry scheme fits the execution plan; tolerance is 1cm at a scale of 1:1.

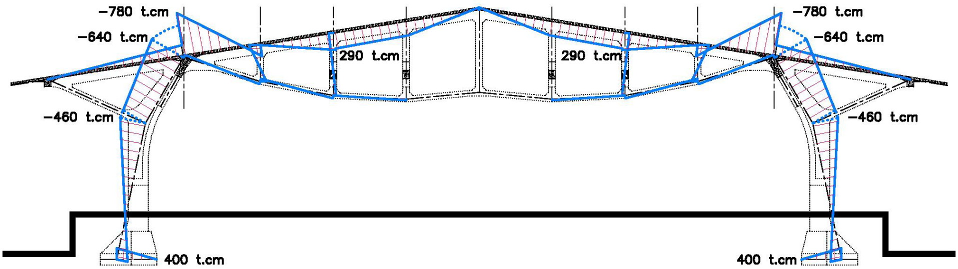

Fig. 22. Chiasso Shed (1924): bending moments are mostly due to deformation compatibility (credit: DZ).

Fig. 23. Maillart's Schwandbach Bridge (1933) – credit: ETH-Bibliothek Zurich, Image Archive/Robert Maillart Archive.

Federica Ottoni

Of domes and their trap. The long history of the domed structures, between debates and experimentation

«Is there any point to which you would wish to draw my attention?»
«To the curious incident of the dog in the night-time».
«The dog did nothing in the night-time».
«That was the curious incident…».
Sir Arthur Conan Doyle, Silver Blaze (1892)

Introduction. The paradigm of the "error"

This research starts from a privileged point of view, the "error", related to a precise constructive form, the dome, with the aim of looking at its solution over the centuries.
The history of masonry domes is quite long and, certainly, the present study is not meant to be exhaustive. Some specific domed buildings and some debates which have burst on them, have been selected in this paper: they can be actually considered as many salient points in the definition of a constructive theory of masonry domes. The final aim, not too hidden, is to demonstrate how these specific case studies have often represented as many accelerations in the complex and slow transition from Art to Science of Building (Benvenuto, 1981b).
In his book, Henry Petrosky says that Imagination and Fear are the most valuable tools for engineers or architects to avert the tragedy; moreover, only those who know their past can lead to a progress (Petrosky, 1994). Therefore, only through the observation of the past it's possible to move forward and the issue is even more interesting for historical construction analysis: construction practice (commonly called the 'Art of building') has reunited in time with the unchanged laws of Statics (the 'Science of construction') only thanks to the cyclic proceeding of observing the 'error' in the ancient monuments (their collapse) and thus re-building them correctly.
We can call this method 'empiricism' and it represents both a methodology and a warning: the understanding of failures plays a vital role in any error-free design and each successful construction can be considered as

the result of a 'pre-vision', adequate and complete, of what would have not worked (Ottoni, Coisson, Blasi, 2009; Ottoni, 2017).
The 'error', is the vantage point for observing the evolution of masonry structures during centuries (Levy, Salvadori, 1997). In the 'correct' ancient constructions (and those arrived to us definitely are) each structural element has a proper role – from the aesthetical, static and technical point of view – matured through the observational method: by finding the solution to constructive 'errors'. The ancient master-builders were certainly unaware of the theory at the base of the structural behaviour of masonry, which slowly developed after centuries of construction practice; nevertheless, they could have experienced the final effect of this structural mechanism, through the simple observation of cracks. With time, a series of subsequent approximations led to the mysterious transition from Art to Science of Building, thus transforming intuition into proper 'understanding' of the solution of the 'error'.
Moreover, a possible interpretative key of the evolution of masonry constructions can be the 'building technique'. The technical knowledge, often, allows to abandon tested and consolidated constructive schemes in order to dare something new, which, at a first sight, does not appear sufficiently stable. Without the technological development registered in the meantime, even the more efficient (and visionary) structural invention would not have been possible, thus determining the scientific advancement (Hempel, 1982).
The paradigm of the 'error' can be Babel. At some point, someone must have thought that the towers would have lost their challenge against the sky. And maybe that's when it was decided to close the sky into domes, better if large enough to contain it (Todorov, 1984).
Once you decide to close the sky into a dome, it's easy to get carried away and this can be seen as a sort of *ubris* which, as in Babel case, has to be punished. Then, inevitably, it happens the collapse, which actually belongs to a peculiar time: the time of Prometheus's liver, which, eaten during the day, regenerates itself at night, thus connecting the "God time" (eternal, in which nothing happens because everything is already there) to the "men time", in which everything simply flows (Vernant, 1999).
The circular "time of Prometheus liver" stays in the middle, as the "error", which refines the main principles of construction in a repetitive cycle of collapses and reconstructions, thus joining the eternal and unchanged laws of Statics (the Science of Building) to practice of building (the human time): by eating the liver, the practice is redirected to the ideal model.

Actually, also the domed structures stay halfway between Gods and men, in an intermediate space which "joins heaven and earth, divinities and mortals" (Conforti, 1997) and the translation from the original monolithic dome to the successive two-shells configuration well represents this divarication of spaces. Indeed, not surprisingly, in the last example of the following selection – the Pantheon in Paris – a third shell inserted in the dome gets into this intermediate space, definitely solving (both in the intermediate time of "error") dome structural problem.
But this is an anticipation of the end of the story I want to tell herein, which deals with the structural principles of domes and their progressive definition during centuries.

The dome "trap": the horizontal thrust

With hindsight, the problem of domes has remained the same over centuries: based on the same thrusting principle of the arches which compose them, domes tend to open themselves, by transferring to their bearing structures, not only vertical forces but even horizontal ones. These thrusts generate severe tensional states, which inevitably, during time, determine their lack of stability. This theory still holds for domes.
A dome, in fact, can be seen as a series of arches obtainable by slicing the structure along meridian planes: two opposite slices form an arch and they work together by compression, making the most of the masonry mechanical properties. Therefore, it is possible to draw, for the dome, the 'line of thrust' whose minimum and maximum values determine the beginning of collapse mechanisms. In this way, it's easy to identify the specific equilibrium state in compression at which the dome is safe **[fig. 1]**.
It's well known that rotational domes, unless particular conditions of loads and constraints, are automatically funicular of their distributed loads. The one just described is the 'membrane regime', which proved to be useful in describing the mechanical behaviour of domes and in providing an intuitive idea of the internal efforts (Heyman, 1977; Heyman, 1982). But this perfect functioning has a flaw, that in case of masonry domes is unavoidable: the states of traction at the base which cannot be eliminated. Then, masonry domes tend to open themselves and transmit to their bearing structures, not only vertical forces but even horizontal ones.
We can call it the 'trap' of masonry domes (Ottoni, 2012).
Vertical fractures have been found in nearly all masonry domes, thus highlighting their fundamental mechanical principle (the arch) as well as their

typical mechanism of instability. Although fractures can vary for both amplitude and position, at the end ‘the trap is done’ and the dome is transformed from a compact and solid body into a sequence of arches with variable sections, mutually contrasting through the compressed parallels.
In the following sections we will retrace the evolution of structural conception of the domes by analysing some fundamental historical debates. In this way it will be possible to identify how the progressive enhancement of technology has converted the original ‘art of building’ into ‘science’.

The ‘proportional theory’ and the technical ‘jumps’

According to the described methodology, this story doesn't follow a chronological order but the constituting episodes are divided into two main series: the “Great book of the errors” fights the visions and the revolutions proposed by the “Paranoid architects” (Zetlin, 1988).
The case studies examined here represent the different failures, collapses and reconstructions which have progressively clarified the possible traps in the design of arches and domes, that only some “paranoid” architects (as Brunelleschi, Soufflot and Rondelet) have been able to solve.
This ’obstacle course’ over the centuries has definitely shown that, despite the fact that the ancient master-builders didn’t possess the modern mathematical instruments (proper of the modern science of construction) they have realized such complex structures, perceiving by intuition a ‘static sense’ which permitted them to correct the previous constructive ‘errors’, just by observing the past (through ‘empiricism’).
The reconstruction of this historical route can represent an additional fundamental instrument for a deeper knowledge of these magnificent historical domes and it can help in clarifying the difficult process from ‘art’ to ‘science’ of construction.
Proportional theory and geometrical rules originated from the direct observation of collapse mechanisms occurred to the monuments during the centuries. The comparison between similar mechanisms and pathologies has allowed to trace the field of application of these theories in solving the safety problem and in guaranteeing the equilibrium of masses. Certainly, ancient architects couldn't have a precise idea of the mechanical behaviour of masonry, but, for sure, they must have figured out the ‘forces path’ inside their huge structures.
Not always these geometrical rules have been codified in proper architectural treatises; most of times – also etymologically – ‘monuments’ are

'documents' of many basic evolutions (geometrical and constructive) in the history of construction. These evolutions have been possible after some 'jumps' (technological and theoretical) in an otherwise repetitive and cyclic history of construction which basically derives from the constant application of well known and consolidated geometrical rules (it doesn't matter if theoretically captured).
The aim of this paper is, at the end, to identify some of these 'jumps' (Como, Iori, Ottoni, 2019).

Some historical debates, from 'art' to 'science' of construction

Since the beginning, the design of domes involved two main problems: 1) to determine the best shape (and thickness), able to contain the curve of descent loads (line of thrust) and 2) building tambours thick enough to counteract the thrusts at the base of the dome. Most times, the solution of the 'trap' has been just the geometry (by the application of proportional method), but only when it has encountered a perfect match with 'error' prevision, mainly with the help of technology, it has been successful.
Few examples, related to either constructions or debates, have been selected here, which can be actually considered key points in the definition of a constructive 'theory of masonry domes'.

The perfect geometry of Hagia Sophia and the solution by empiricism

Hagia Sophia is the first dome of this story. In this case, the answer to the 'trap', originally given by its perfect geometry, failed. Emperor Giustiniano called two '*Mechanicopoioi'* (and not simply architects) in order to build his divine sphere and, despite their knowledge and the use of a perfect geometry, they challenged collapse at least three times during construction phases. The two mathematicians had to cover a great space with a symbolic shape, conciliating the roof implant with the longitudinal plan and they did it, by elevating the divine sphere of Pantheon through pendentives (*pennacchi*).
Then, the hidden trap of the dome had to be solved, by finding the way of contrasting the thrusts and two solutions have been applied, following the functional and structural dualism between plan and elevation: two massive buttresses in North-South direction and two semi-domes in the East-West direction, both organized in a rigorous geometrical structure,

known since ‘70s[1] (Mainstone, 1969; Sanpaolesi, 1978) and confirmed by the topographic survey of 1996[2] (Sato *et al*., 1996; Blasi, Bianchini, 2001). Twenty years after the construction, in 557 BC, the first dome – with lowered profile and probably with circular tambour – collapsed after an earthquake; the second dome, subsequently reconstructed by Isidoro di Mileto, constitutes a geometrical correction of the first ‘error’: the profile of the dome, by the architect, who attributed the cause of the collapse to the profile of the dome (too lowered). Nevertheless, the dome collapsed other two times (in 989 BC and in 1346 BC), always after an earthquake, thus definitely clarifying the inability of this monument to cope with seismic events (Blasi, 2003).

Constructive ‘errors’ in this case had double origins. The first one was technological and it dealt with materials and with the byzantine building practice which foresaw the use of high thickness mortar: the high ratio between mortar and inert (nearly 1), increases the intrinsic deformability of masonry, strongly affecting its resistance (Livingstone, Stutzman, 1993; Binda, Tedeschi, Baronio, 1998). The second error was geometrical and strictly connected to the inhomogeneity, along longitudinal and transversal directions, in counteracting horizontal forces: on the eastern and western sides, dome thrust was actively counterbalanced by two semi-domes, whereas the great buttresses on northern and southern sides were not effective during earthquakes (Mainstone, 1965). Moreover, while semidomes can exercise their constraint effect even once deformed, the great pillars rhythmically move during earthquakes, separately from the rest of the structure, thus not contributing to the dome stability **[fig. 2]**.

Only a thousand years later, Sinan solved the ‘mistake’ by straightening the pillars at their base and by encircling the dome with huge iron ties[3] and,

[1] See, in particular, a letter from Rowland Mainstone to Raybun Taylor, published in *The Journal of the Society of architectural historians*, vol. 55, n. 3, september 1996, pp. 360-362, and Sanpaolesi, 1987, pp. 135-136, n. 1.

[2] A precise topographic survey of Hagia Sophia has been carried out in 1996 by the Japanese group of research leaded by T. Sato, K. Hidaka, Y. Kawabe, T. Aoki and K. Yamashita, and the results can be seen in Sato *et al*., 1996.

[3] Sinan understood the errors of Hagia Sophia and he applied the same solution to its trap in his two replicas of this magnificient dome (Suliman Mosque and Blue Mosque), repeating its perfect geometry and the most effective system (the semi-domes) on both the main directions, together with the encircling tie-rods, which in Hagia Sophia are actually

in 1847 the Swiss architect Gaspare Fossati, charged of the restoration of Hagia Sophia, added two more orders of tie-rods, one at the base of the dome and the other one in the tambour; furthermore, he built some rampant arches, now removed. We do not have any evidences of mathematical calculations made by Sinan but we can certainly recognize the first overcoming of the ancient geometrical rules made by empiricism. In keeping with this, encircling tie-rods remained, at least until XVIII century, the traditional method for dome strengthening (Ottoni, Blasi, 2016).

The great Vatican Temple: memories between mathematics and esperimental physics

It's hard to fix a precise date for the entrance of Maths and Physics in the structural issue of domes. Even more difficult is to establish when exactly they have substituted the empiricism.
Perhaps it was during the great debate on Vatican dome, which hides the battle between Galileo, and the new experimental science, against the traditional geometrical rules (not only the proportional ones) and sees "three Matematicians" against the physician Giovanni Poleni (Niglio, 2007).
Some years before, Carlo Fontana (Fontana, 1694) had described the cracks and damages of the great dome, with the final aim of describing, in his treatise, the most suitable rules of the ancient architects which would demonstrate the supremacy of the Art of building on the new Science[4] (Hager, 1992).
Although the alarm concerning the first cracks on the great dome dates back to 1603, only in 1740 Pope Benedetto XIV decided to nominate a Scientific Commission, composed by "three Mathematicians": Giuseppe Boscovich, Tommaso Le Seur and Philippe Jaquier (Le Seur, Jacquier, Boscovich, 1742). After Galileo, empiric answers to static problems were no more sufficient, and general theories and incontrovertible judgements were needed (Baggio, Da Gai, 2000).

hidden into masonry but which are visible in the other two monuments.

[4] Actually, Fontana had failed, just highlighting the dimensional ratios of Saint Peter in which the proportional rules had been twisted: the tambour measures only 3 m thick, rather than the 4.2 m (upgradeable to 4,7, equal to 1/9 or 1/10 of the internal diameter) prescribed by the treatises.

At that time, the dome was no longer a monolithic system: the parallel continuity was interrupted, from the impost up to the lantern, thus transforming the dome into a series of arches, connected only at the top by parallel band. Moreover, in San Pietro dome, a correlation between the deformation of the tambour and the vertical propagation of the cracks along the meridians was evident. As anticipated, domes tend to open themselves, splitting into segments – arches – in which the funicular of loads does not coincide with the median; this fact provokes horizontal thrusts at the impost which the ancients try to fight with thick walls or with double shells stiffened by ribs. The problem lies always in the balance between two forces: the force of the active work of dead weights, which lowers the central area, and the passive work of the lower bands which, conversely, tend to rise up. In the debate on San Pietro, both issues are analysed for the first time: the funicular of loads by Poleni and the balance of forces by the Mathematicians.
The second ones applied for the first time the Virtual Work Principle to the system composed by the huge dome and the underlying tambour, reaching the correct visualization of movements of each single element involved in the collapse process, almost by anticipating the concept of macro-elements. They were concerned about the horizontal cracks, and by the detachment between the two spheres, which was evidenced by the cracks along the corridor between the two domes. They considered the hinges around which the un-cracked masonry (rigid and un-deformed) had rotated, causing the movement of the factory. It is important to emphasize that, in their analysis, they did not divide the dome, as commonly accepted, into subsequent arches; by contrast, they considered the dome as a whole, thus applying for the first time the 'principle of virtual works' and the concept of 'work of deformation', despite some uncertainties in terminology **[fig. 3]**.
It is beyond the aim of this work to discuss in detail the results of their calculations but it's important to emphasize that their analysis represented the birth of 'the scientific method', aimed at scientifically explain the observed effects of hypothesized mechanisms (loss of verticality and cracks) and the underlying causes (movements of each part, with associated diagrams and static schemes)[5]. They don't have yet the knowledge of the distributed

[5] Recent calculations have shown that with only 6% weight more, the structure would

loads and they hypothesized that the weight of the lantern had produced the damage and the deformation of the tambour with the enlargement of the circumference at its base. The real breakthrough lies in their method, despite some flaws such as the work of expansion of the rings which was (wrongly) calculated as in the case of a straight chain (Como, Iori, Ottoni, 2019). Actually, they split the structure into ‘macro-elements’ (*ante litteram*), then considering their mutual relationship[6] (Di Pasquale, 1979).
Completely different was the Poleni’s approach: he was a physicist, mainly focused on evaluating the quality and the characteristics of materials. Starting from the observation of cracks, Poleni divided the dome into segments-arches, thus leading to the inevitable passage from the single chain (traditionally used for arches) to the encircling tie rod system (Lopez, 1957; Di Pasquale, 1994).
He was firmly convinced that dead weight was crucial in endangering the dome stability; moreover, by anticipating later debates on the France Pantheon, he identified the interaction between materials as the main cause of cracks in the tambour (the travertine, stiffer, takes more weight, thus reaching the breaking value). Despite his still animistic language (the materials are referred as living bodies with an internal ‘juice’, which, once iced, will split the stone[7]), he had a great intuition and he experimentally determined the breaking value of iron creating a *machina divulsoria* able, for the first time, to test the strength of materials.
In 1748 Poleni stated in his memoirs (Poleni, 1748) that the dome was stable and showed static graphics and experimental tools to support his theory: he divided the dome into 50 arches, each further divided into 16 ashlars; in addition, in accordance to the recent Hooke’s theory, he demonstrated that the funicular line was entirely contained into the thickness of the arch, thus proving its stability[8].

have collapsed. See Como, 1997.

[6] Years later, in the discussion about the cathedral of Milan, Boscovich will apply the same principle, but, this time, he will consider also the friction.

[7] The experimental reference goes to the experiments by Musschenbroek and Gravesande on iron.

[8] Actually, in the mid-700’s, the theory on the arches is refining but Poleni, although he has the mathematical tools to achieve the final curvature of the cable by means of analytical approach, uses the graphical method, and that’s what Di Pasquale and Navier will reproach to him (Di Pasquale, 1979).

It is now evident that Poleni anticipated the ‘safe theorem’ (Heyman, 1982): the acceptable equilibrium of an arch could be realized only if internal stresses would be of compression. Nevertheless, he disregarded the tambour in his analysis, thus underestimating the real level of damage of the dome (Baggio, 1990; Como, 1997).
It’s curious that, after demonstrating the stability of the dome, he prescribed some encircling tie rods in order to stop crack evolution. We can reasonably attribute this prescription to the Architect Giuseppe Vanvitelli, who used the proportional theory for “calculate” the possible collapse of the dome, which didn’t fit with the proportional rules given by the ancient treatises (the tambour thickness didn’t reach the prescribed 1/10 of the diameter of the dome) and, for this reason (before any structural calculation and material analysis) would have collapsed (Di Stefano, 1980; Mainstone, 2003).
For this reason, the episode of Saint Peter rightly stays in the ‘great book of errors’, since the solution of the encircling tie rods finds in the past, rather than in Science, its own justification.
It’s a matter of fact that, between 1743 and 1747, Vanvitelli himself oversaw the works for the insertion of five encircling tie rods in the dome; in 1748 the strengthening intervention was completed, the dome was finally stabilized and the voices of alarm definitely allayed (Brusatin, 1971).

Brunelleschi, “paranoid” architect, and his tricks against the trap

Certainly, the great architect Filippo Brunelleschi was “paranoid” and may be for this reason Santa Maria del Fiore dome, although inspired by Hagia Sophia, differently from that has remained unchanged over centuries and stands unique, among masonry domes, to be devoid of iron encircling systems.
Brunelleschi adopted the particular slack cable (*cordablanda*) disposition for masonry (obtainable by the intersection between cones and cylindrical surfaces) in order to avoid any disruption in the corners of its octagonal dome, recalling the constructive process of rotational domes. The main aim was to avoid the creation of weak areas in the angle spurs, which – also intuitively – had a key role in the dome stability (Chiarugi, 1984; Fanelli, Fanelli 2004; Dalla Negra, 2021).
The second trick invented by Brunelleschi was the herringbones (*spinapesce*): this technology consists of a series of bricks interrupted at regular intervals (about 1.20 m) by vertical, radial oriented bricks, which have

the function of containing the masonry during the building process of the dome without the use of scaffoldings (*centina*); in addition, even more important, they create a dome of radial propellers in the thickness of the walls (Fanelli, Fanelli, 2004; Pizzigoni, 2014). These spirals actually follow the isostatic lines inside the shell, which are always tangent to the principal directions of stress. Brunelleschi, although unaware of the underlying scientific theory, certainly foresaw this benefit and thus he was able "to prefigure the impossible" (Rossi, 1962; Sanpaolesi, 1951).
More important for the purpose of this discussion, is the central debate on the cracks (*screpoli*) of this magnificent dome which highlights the importance of this structure and of the technical accomplishments used in its construction.
Some centuries later, in 1694, Vincenzo Viviani, the Mathematician of Tuscany Gran Duke, climbed the dome stairs and something new happens. In those times, some cracks (*screpoli*) were noticed on the great dome, and Viviani (as member of a Scientific Commission - composed by his Mathematician together with Giovanni Nelli, Filippo Sengher, Guerrino Guerrini and Gian Battista Foggini was charged to determine, through tests and experiments, whether cracks were sensibly enlarged in times (Galluzzi, 1977; Barbi, Di Teodoro, 1989).
On that occasion, the dome was equipped with a primeval monitoring system, consisting of marble blocks and iron wedges able to evaluate the dilation of the cracks over time. One year later, all spies were deformed and some were broken[9]; therefore it was clear that the dome was still moving and finally, on June 20th, 1695, the final solution of the 'error' was deliberated: the dome had to be encircled by four orders of iron chains (Nelli, 1753; Di Teodoro, 2011; Ottoni, 2012).
It must be considered, however, that this solution was not 'innovative', as we have seen: at the end of '600, in fact, it was already well known that the same collapse mechanism was common to all domes, e.g. the lowering of the top with the generation of lateral thrusts at the impost; as a consequence, the insertion of hooping chains was a shared remedy[10].

[9] Drawing by Federico Fontana, in Fontana, *Accesso ultimo fatto sopra la Cupola di S.M. del Fiore…,* Firenze, Biblioteca Nazionale Centrale, ms. II_21, c. 94.

[10] In step with this, we can recall here Baldinucci's words: «There is no architect, although incompetent, who doesn't know that chains are needed to make a so large dome» (non

However, the unexpected occurred: Grand Duke received an anonymous letter (later on attributed to the architect Alessandro Cecchini), which undermined at its base the Commission conclusions: the opening movement hypothesized for the dome was considered 'absurd' and the cause of collapse had rather to be searched in differential settlement of foundations. As a result, not only the proposed chains were declared unnecessary but moreover, although necessary, they would be insufficient, because too slim (Di Teodoro, 2021).
Thus, Viviani was forced, unwillingly, to defend his theory and, for the first time, he tried to mathematically demonstrate the effectiveness of the hooping chains: through a quite complex trigonometric formula, Viviani finds the relationship between the weight of the single element and the horizontal thrust at its base, referring to the simple formulation already applied by Torricelli in the Uffizi '*columnae fessae'.* This was the first mathematical formulation of a generally accepted intervention, despite it was obtained without a real connection to dome behaviour. The passage from *vectis* to *trabis* and then to *columna*, can be retraced and explained in a letter addressed by Viviani to Michelangelo Rizzi, in which, through a quite complex trigonometric formula, Viviani finds the relationship between the weight of the single element and the horizontal thrust at its base[11] **[fig. 4]**. Despite the production of the iron chains had been stopped, for the first time the issue of tie dimensioning was mathematically approached.

Soufflot, Rondelet and the sublime intuition

The problem of horizontal thrust has been then solved in the last debate on the French Pantheon, during the XIX Century; it has decreed the end of 'masonry domes' – as they were originally conceived – and the parallel birth of modern 'membranes', definitely translating the original equilibrium approach into a resistance problem.
Soufflot's revolution in designing the dome of Sainte Genevieve was Mainly based in a number of exceptions to the traditional geometrical rules

ci fu mai architetto, per debole che fusse, che non sapesse che ci volevano le catene per fare una cupola di cosi grandi dimensioni), in Ludovici, Samek, 1948, p. 167.

[11] Manoscritto Galileiano 222.c.120r, Firenze, Biblioteca Nazionale Centrale di Firenze.

made by the architect, who skilfully mixed Gothic insights with the new technological possibilities (Braham, 1971).
We do not know if, at the time of the project, Soufflot already knew how to solve the treachery of the dome and how to definitely transform it into something else, or whether Rondelet had introduced the innovation of materials. But it's not important here to determine who finally solved the dome's trap. What really matters is that the great debate (or rather two) on the French Pantheon, was recognized as a great acceleration in the structural knowledge, thereby allowing the final understanding of the constructive principles of domes.
Two are the well known debates aroused around the French Pantheon, but the issue is always the same: the thrust of the dome, which in the size of the pillars finds the pretext for the discussion (Blasi, Coisson, 2006a).
Actually, the present dome is much larger than the first one designed by Soufflot and the reasons for the performed changes have to be retraced in both constructive difficulties and related controversies.
Since Soufflot was very well aware that slender pillars would have not endured a too pushing dome, he firstly chose a geometric solution to the problem, considering the previous 'trick' used by Wren in London (Bennett, 1982). In 1757 Soufflot designed a hemispheric, single shell and thin dome, in order to limit thrusts. Thereafter, in 1764, he changed it into a truncated cone which invoked the *tholos* (yet recalling Wren) completed by a dome with a lantern at its top in 1770 (and definitely in 1777). The current system of three superposed domes, whose stability is to date still in a good state, were built by Rondelet in 1790, when the debate on the monument reached the crucial point.
This system is composed of a lower circular dome, with a central oculus from which it was possible to see the outer shell, also spherical, whose diameter measures 23 m. The lantern lays on the intermediate dome, which has the perfect – and more efficient shape of a catenary (thus recovering the recent Hooke's theory).
Despite the geometrical and technical tricks (the system of domes) and the perfect shape of the bearing structure (the catenary), the problem of reducing the pushing of the dome on pillars still remained. Rondelet faced this problem with technology, thus transforming masonry into armed stonework[12] (Rondelet, 1779).

[12] Maybe, also in this operation he actually follows Soufflot's indication. Infact Gauthey,

The deep and precise geometric knowledge (*catenaria*), joined to the innovative use of materials (reinforced stone) was thus able, 16 centuries after the first Roman Pantheon, to definitely solve the trap of the dome. Moreover, the final structure of French Pantheon is also revolutionary because it hides its load-bearing capacity, dissimulating it into its slenderness, offset by new technologies and materials used.
Curiosly, exactly as for Brunelleschi and his tricks against the dome trap, Soufflot (and Rondelet) could not foresee that the inclusion of iron into the stones would have provoked in time, due to the hygrothermal oxidation, damages to the structures (Blasi, Coısson, Iori, 2008).
Indeed, the current crack pattern of the monument testifies two different phenomena: subsidence problems and oxidation phenomena of the iron inserted into the stone blocks (Blasi, 2005).
Anyway, the dome is not involved anymore in its collapse mechanism, due to the lack of pushing mechanisms. The Soufflot's revolutionary design of slender pillars have challenged the ancient proportional rules and the final covering of the new laic temple has solved in its structural configuration the problem of the thrust (Patte, 1769; Gauthey, 1789).
A new material – made by stones and iron links – was finally capable to absorb the ineradicable tensile stresses at the base of the great dome, thus justifying the verdict of Rondelet: "domes don't push anymore" (Rondelet) **[fig. 5]**.
In conclusion, the two visionary architects, Soufflot and Rondelet, not only minimized thrusts through a deep geometrical knowledge, but they also encircled the dome with hidden heavy ties, transforming it into a rigid body, which was able to transmit only vertical loads on pillars (Heyman, 1985). This can be considered the end of the centennial debate on domes and their stability, reached thanks to a mix between technology enhancements and scientific intuition.

The curious incident of the domes in the night-time: possible epilogue

Until XVII century, the problem of stability of arches (and domes) had been solved through proportional rules on the ratio between thickness and

in 1771, underlined that the iron encircling the dome would have given a solid consistency to the dome itself, in this way anticipating the modern concept of "membrane".

span (or diameter), aimed at reaching equilibrium states. In the meantime almost all the domes had begun to show the signs of their weakness, or, we can say, their trap: their horizontal thrust (Pini, 1770).
The paradigm of Babel (the “error”) was experimented several times before it was understood by the architects (visionary or paranoid) and it has been translated into the numerous debates on dome stability, before coming to a conclusion. The history of construction can be ultimately seen as the translation of shape in structure: the final shape of a construction used to be the formal result of an empiric calculation – strict and scientific: each built monument constitutes an experiment, 1:1 scale, whose positive, or even more negative outcomes (as repeated collapses) has given in time important information for the correction of the ‘error’ (Giuffrè, 1982).
Starting from the two *mecanicopoioi* of Hagia Sophia, passing through Brunelleschi’s terrific inventions, to Soufflot’s slender design of Sainte Genevieve in Paris, the question of the ‘dome’s trap’ (its horizontal thrusts) has been progressively understood and the examples here examined have progressively clarified the ‘error’ in the design of domes. Only some ‘paranoid’ architects (such as Brunelleschi, Soufflot and Rondelet) have been able to overcome, mainly through technical tricks, the error. This obstacle course during centuries, has forced architects and scientists to find new scientific and technical approaches to fight the centennial war against the ‘dome trap’, progressively abandoning the ancient ‘proportional theory’ moving towards the principles of modern ‘science of construction’. These debates have impressed as many accelerations in solving the ineradicable constructive problem of the domes **[fig. 6]**.
At the end of the story, we can say that the reduction of the stability problem into an equilibrium one works, at least until we are far away from the characteristic values of rupture for the materials and the equilibrium problem provides a key to unlock the secrets of the structural behaviour of the domes, which the ancients knew well. Then, Galileo arrived, who can be considered the founder of the newborn “theory of structures”, not only considering and calculating, for the first time, the strength of materials but also discovering a new design rule (the square-cube law), which was fully contradicting the traditional approach (the proportional design).
But, ultimately, we can say with Santiago Huerta that “Galileo was wrong” in applying this argument to ancient masonry structures (Huerta, 2006).
What Galileo didn’t consider was that, for a particular range of sizes and strengths, typical of historical structures, in which the limit resistance of the material is never reached, the respect of proportional theory could

actually ensure the overall stability of the construction, precisely by virtue of their great size.
Indeed, the proportional theory had been applied to structures which were working well below their strength capabilities and, in virtue of this, it can be considered still valid for the particular field of ancient buildings, thus translating the problem of resistance in a stability one (Fig.20) (Como, Iori, Ottoni, 2019).
It is a fact, in fact, that for historical masonry structures, the stresses are an order or two orders of magnitude below the ultimate strengths of the masonry and, therefore, the problem of masonry design is not governed by strength but by stability, which the ancient master-builders have demonstrated to know very well (Giuffrè, 1982; Giuffrè, 1991; Benvenuto, 1981b; Di Pasquale, 1998).
In other words, if proportional design is applied to buildings with a limited scale ratio, the balance equation satisfaction is also guarantee of not overcoming the material ultimate resistance.
The approach of the ancient masterbuilders, or better their "Art", seems to be successful in perceiving by intuition and in solving the hidden theachery of masonry domes. Finally, this is the "curious incident" occurred to the masonry domes during centuries: they seem to be cristalized in the geometrical study, since, at the end of XX century, Jacques Heyman returned to Poleni (Heyman, 1988), in order to demonstrate the efficiency of the static graphic instruments in finding the equilibrium of vaults and masonry arches. Indeed, the safe theorem and the limit analysis, after three centuries, returns to "geometric factor" to reliably verify the stability of an arched structure, and a dome (Heyman, 1998; Block, Ochsendorf, 2007).
Only apparently because, as we have already seen, in the meantime, the mechanics and the modern "Science of construction" have justified their functioning principle, and, thus, their mechanical understanding.
Despite mechanical behaviour of domes (i.e. their lack of stability due to horizontal thrusts) couldn't be mathematically demonstrated by ancient architects, we can find more and more correct theoretical approximations of the solution to the 'dome trap' by examining some successive domed monuments.
The structural conclusion, indeed, is always the same: the constructive problem of masonry arches is essentially geometric and one of the most important consequences is the equilibrium approach, which can be used as analysis instrument for the ancient structures, extensible also to the domes, also considering their strengthening and conservation.

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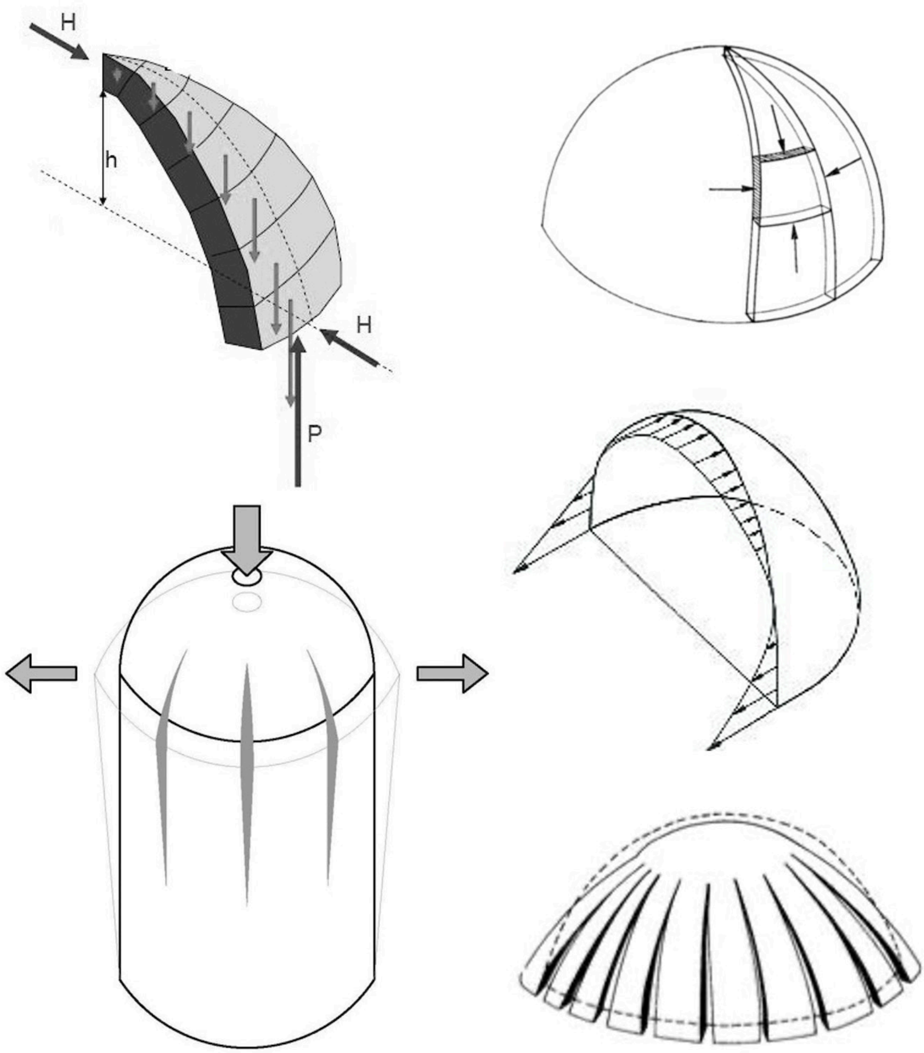

Fig. 1. On the left, the well known mechanism of collapse (the “trap”) of masonry domes which, under self weight, tend to open themselves. On the right, the schematization of internal stresses (Heyman, 1969).

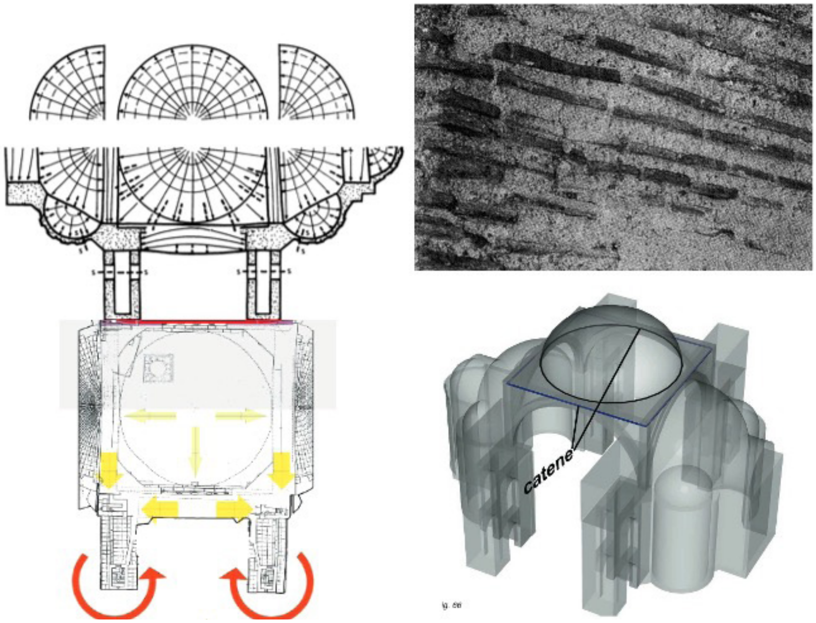

Fig. 2. On the left, the double system of counteracting the thrust of the dome (pillars and semi-domes) and their weakness; on the right: (above) the high ratio between mortar and inert, (below) the two orders of chain inserted by Sinan and Fossati.

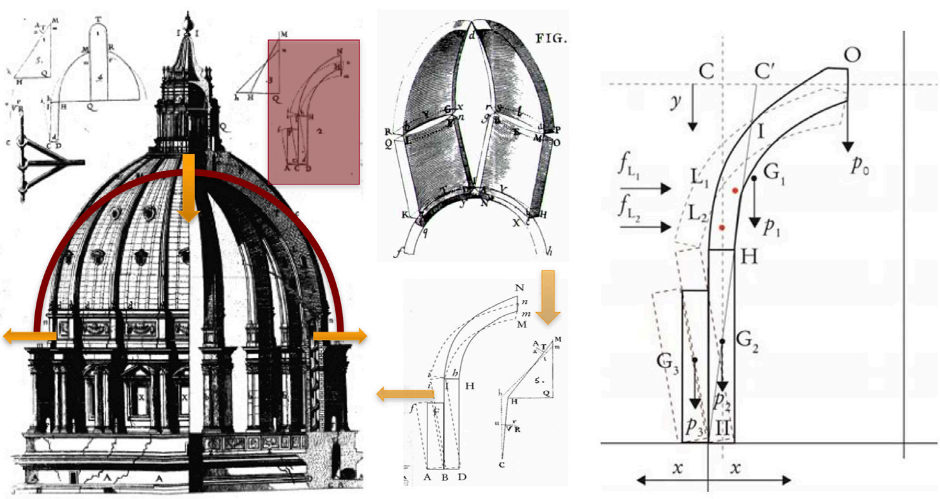

Fig. 3. The analysis made by the Three Mathematicians, which anticipate the “macro-elements” and the “virtual work principle” (on the right, the modern cinematic analysis (Como et al., 2019).

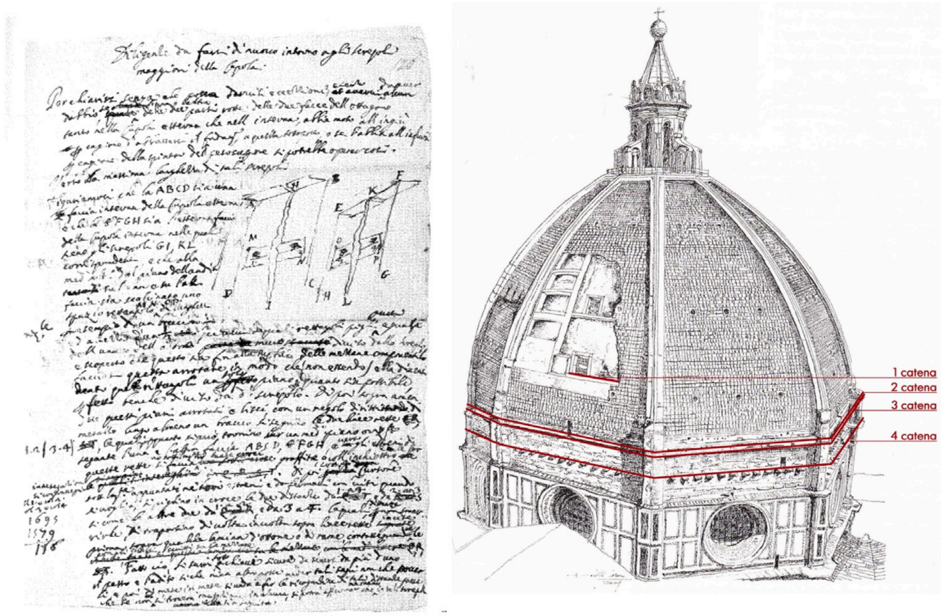

Fig. 4. On the left, the first calculation of chain, made by Vincenzo Viviani (Galluzzi, 1977) and, on the right, the four orders of chain prescribed by the Gran Duke Commission.

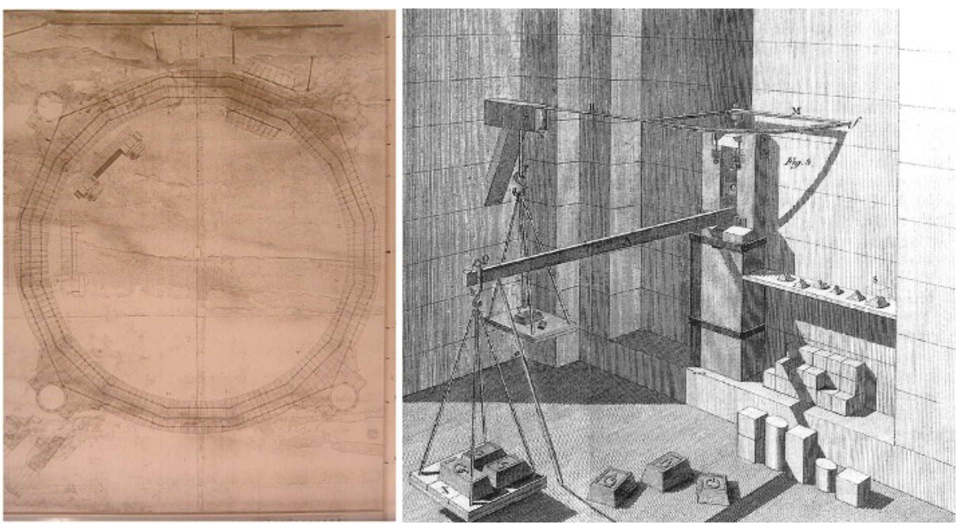

Fig. 5. On the left the armed masonry of the dome, which “doesn’t push anymore”, and, on the right, the first test on materials made by Gauthey (Rondelet, 1833).

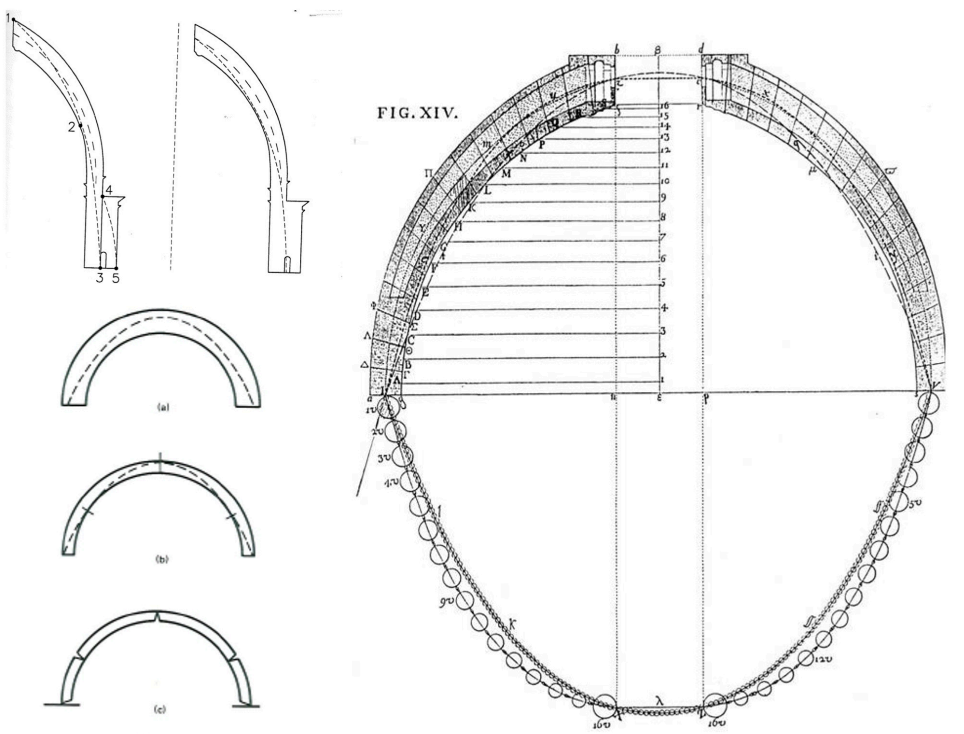

Fig. 6. On the right, the Poleni's analysis of one of San Peter slices, by Hooke theory; on the left (above) the correct visualization of the line of thrust (Como, 1997) and (below) the safe theorem (Heyman, 1966).

Marzia Marandola

Autobiografia della ricerca

Il volume premiato[1] ha costituito una tappa miliare nel mio percorso scientifico e accademico: grazie ad esso ho ottenuto una significativa affermazione nell'ambiente scientifico e una riconoscibilità disciplinare, che mi ha avvantaggiato nelle relazioni universitarie in Italia e all'estero. L'originalità del tema affrontato nel volume e il taglio critico innovativo con cui è stato trattato, sono stati determinanti per aggiudicarmi nel 2011 il posto da ricercatore a tempo indeterminato in Storia dell'architettura alla facoltà di Architettura della Sapienza, Università di Roma. Il libro ha costituito un vero e proprio viatico per i miei studi, rivolti a sondare un terreno relativamente vergine che, pur trascurato dalla critica architettonica, è provvisto di vigorose valenze tecnico costruttive e figurative. Questa visione duplice, che sovrappone l'opera di ingegneria e quella di architettura, imprime un'innovativa trasversalità disciplinare al volume. Il medesimo ambito si è attestato come privilegiato nei miei studi successivi, che hanno spalancato nuovi territori alla critica architettonica, fino ad allora molto reticente, se non estranea, nei confronti delle costruzioni "di ingegneria".

Attraverso le modalità costruttive ricorrenti nelle opere in precompresso e il loro funzionamento statico il libro affronta gli esiti formali ed espressivi specifici di alcuni capolavori della costruzione in Italia. Se il filo rosso della narrazione prende le mosse dalla nascita del cemento armato precompresso e ne segue la diffusione in Italia, il fuoco critico si incentra sul rapporto tra le valenze costruttive, di cui dispone questa tecnica innovativa, e le sue inedite potenzialità espressive, plastiche e formali.
Antefatto di questo studio ad ampio raggio cronologico e tipologico è la ricerca di dottorato che ho dedicato a *Riccardo Morandi ingegnere (1902-1989): le sperimentazioni e le opere in cemento armato precompresso degli anni cinquanta* (XVIII ciclo, Università di Roma Tor Vergata, tutor prof.

[1] M. Marandola, *La costruzione in precompresso. Conoscere per recuperare il patrimonio italiano*, Milano, IlSole24ore, 2009, libro vincitore della Menzione speciale Edoardo Benvenuto 2010.

Claudia Conforti, coordinatore prof. Sergio Poretti) condotta e discussa nel Dipartimento di Ingegneria Civile dell'Università di Roma Tor Vergata, dove dagli anni Novanta del secolo scorso, sotto la spinta degli studi di Vittorio de Feo, si sviluppano studi pionieristici sull'architettura confrontata con la storia della costruzione e quella del cantiere, dal mondo antico fino agli edifici contemporanei.
La tesi di Dottorato ebbe un significativo riconoscimento nella segnalazione al Premio Nazionale AAA/Italia 2006 dell'Associazione Nazionale Archivi di Architettura Contemporanea, nella sezione *Analisi storico-critica a partire da un archivio di architettura moderna e contemporanea* (convenzionalmente dal 1870 ad oggi), che elogiò la mia ricerca «per la rigorosa struttura metodologica nell'affrontare l'archivio e la ricerca tecnologica».
La struttura del volume è ramificata e transdisciplinare, ponendosi l'obbiettivo di fornire strumenti di valutazione e dispositivi di lettura, tali da conquistare alle opere cosiddette di ingegneria l'apprezzamento critico e la densità rappresentativa, di norma riservati alle opere di architettura. Lo sguardo orizzontale è introdotto dallo scritto di apertura del volume, che intende familiarizzare il lettore con le tappe fondamentali con cui si plasma e si afferma la nuova tecnica costruttiva in cemento precompresso. La comprensione di questa tecnica è il grimaldello per introdurre la produzione ingegneristica nella sfera della valutazione critica finora riservata all'architettura e per assaporare una conoscenza che approda a formidabili manufatti plastici, di grande potenza espressiva.
Non casualmente l'immagine di copertina è il viadotto sul Polcevera, un capolavoro costruttivo, paesaggistico e plastico, che ha avuto pochi termini di confronto. I capitoli del libro mettono a fuoco una decina di casi studio, nei quali la costruzione configura sorprendenti equilibri tra originalità formale e arditezza tecnica. Non propone pertanto un semplice catalogo di opere italiane, ma una selezione particolarmente mirata di esempi, nei quali la volontà estetica si intreccia con l'elaborazione del progetto costruttivo, dove progettisti di grande talento hanno scelto, tra le soluzioni tecniche possibili, quella più elegante e figurativamente più raffinata.
Sotto il profilo storiografico occorre notare che l'eccezionalità di queste opere dell'ingegneria italiana è, tra gli altri, prova della volontà di dimostrare che l'Italia, pur uscita sconfitta e impoverita dal secondo conflitto mondiale, pur arretrata industrialmente e tecnologicamente, con cantieri artigianali e tradizionali, è dotata di intraprendenza imprenditoriale e una sapienza tecnica di antico retaggio, tali da approdare a risultati esemplari. Alcune occasioni in particolare hanno creato le premesse per una speri-

mentazione diffusa e audace: la costruzione dell'autostrada del Sole, le opere per le Olimpiadi di Roma del 1960, l'Esposizione di Torino del 1961 per il centenario dell'Unità d'Italia: cantieri artigianali da cui sono uscite architetture esemplari e uniche per originalità e audacia costruttiva. Opere che è imperativo conoscere al fine di conservarle adeguatamente. Molte di esse, soprattutto le opere infrastrutturali, infatti soffrono di un forte degrado e di ammaloramenti diversi, dovuti sia alla progettazione pionieristica che, soprattutto, all'attuale ambiente fisico divenuto particolarmente aggressivo e a condizioni di traffico decisamente esorbitanti quelle di progetto. Si tratta dunque spesso di opere a rischio, come dimostra il caso drammatico del viadotto (1960-1967) sul torrente Polcevera di Genova, che in passato era stato periodicamente minacciato di demolizione dall'Anas a causa della sua sopravvenuta inadeguatezza al traffico attuale e delle difficoltà di manutenzione e monitoraggio. Periodicamente rappezzato, in assenza di un progetto definito e globale di salvaguardia, è tragicamente crollato il 14 agosto 2018, causando la morte di 43 persone. Una tragedia causata dall'incuria, dalla scarsa manutenzione e dall'ignoranza di un'opera sperimentale e avveniristica, come questo libro testimoniava con quasi un decennio di anticipo rispetto ai tempi, ponendo non a caso proprio il viadotto di Genova in copertina, come un caso esemplare di opera da conoscere per recuperare.
Il libro si conclude con un'intervista all'ingegnere Franco Levi, un maestro, classe 1914, morto nel settembre 2009, tra i più brillanti allievi del leggendario Gustavo Colonnetti, caposcuola teorico e garante scientifico della costruzione italiana in precompresso. Levi, con vivacità ed entusiasmo, ripercorre la nascita e l'evoluzione del precompresso, attraverso conquiste e sconfitte di una tecnica ancora oggi in evoluzione. Una sintetica bibliografia ragionata chiude infine il volume, che è stato recensito anche sulla rivista internazionale "Casabella".

La costruzione in precompresso

L'originalità del testo consiste anche nell'indagare storicamente un tema tecnico trascurato dagli storici dell'architettura, quello appunto della costruzione in cemento armato precompresso, spesso lasciato esclusivamente agli ingegneri o ai tecnici della costruzione. In realtà questo sistema costruttivo permette la realizzazione di tante opere del Novecento, tra cui molti capolavori, come il mio saggio dimostra, ripercorrendo la nascita e lo sviluppo di questo sistema.

Dagli anni Trenta del Novecento, dapprima in Francia, poi in Germania, comincia ad affermarsi l'uso del cemento armato precompresso, sperimentato in precedenza in poche e piccole costruzioni.
La paternità di questo procedimento costruttivo spetta al grande ingegnere francese Eugène Freyssinet (1879-1962), che fin dal 1928 deposita un primo brevetto in Francia e, a distanza di un anno, anche in Italia.
Nel nostro paese il merito della diffusione del calcestruzzo armato precompresso si ascrive al brillante ingegno teorico di Gustavo Colonnetti (1886-1968), ingegnere e matematico torinese, docente in diverse università italiane e, dal 1936, direttore del centro studi sui materiali da costruzione del CNR. Pioniere della teorizzazione del cemento armato precompresso, Colonnetti imposta le leggi fondamentali che regolano gli stati di coazione, ossia le cause capaci di indurre la presenza di sforzi interni anche in assenza di carichi esterni esplicitamente applicati.
Un esempio addotto tradizionalmente per chiarire il concetto è quello di un anello metallico di cui, troncata una porzione, si saldano le due estremità rimaste libere, richiudendo a forza la forma: per questa via all'interno dell'anello si genera quello che si definisce uno stato di coazione. Un ulteriore esempio è quello di una trave continua su più appoggi, uno dei quali subisce un cedimento: in entrambi i due casi evocati si generano forze dovute a una presollecitazione. Nel cemento armato semplice si opera una suddivisione degli sforzi tensionali: da un lato il cemento, che assorbe gli sforzi di compressione, dall'altro il ferro che assorbe quelli di trazione. Invece nel caso del precompresso la presollecitazione del conglomerato cementizio mediante l'applicazione di una precompressione, prodotta con l'uso di armature metalliche poste in tensione, gli conferisce la capacità di resistere anche a sforzi di trazione.
Colonnetti ha ripetutamente sollevato riserve sul cemento armato, causate dalla diffidenza verso l'ipotesi di perfetta aderenza e di buona collaborazione tra due materiali così diversi, tra armature e calcestruzzo. Le sue ricerche hanno orientato gli studi di allievi e collaboratori verso la messa a punto di un sistema costruttivo capace di rendere omogenei i due componenti, calcestruzzo e ferro, incoraggiando per questa via gli ingegneri a sperimentare in tale direzione. Gli studi teorici di Colonnetti, seguiti dalla messa a punto di un brevetto, depositato nel 1941, alquanto rudimentale, pubblicizzato ma non commercializzato, sono stati determinanti per lo sviluppo in Italia della tecnica del cemento precompresso.
Tra gli allievi più promettenti di Colonnetti emerge Franco Levi (1914-2009), che con lui collabora prima a Torino, poi in Francia, dove si rifugia

a seguito delle leggi razziali. La collaborazione tra i due prosegue anche in situazioni di emergenza, quali furono gli anni di esilio cui fu costretto Colonnetti, a causa del suo dichiarato antifascismo. Rifugiatosi in Svizzera nel 1943, Colonnetti vi avvia una università italiana per rifugiati: una scuola dove gli studenti fuggiti dall'Italia possano proseguire gli studi. In essa si formeranno alcuni dei suoi allievi divenuti celebri ingegneri, specialisti del cemento armato precompresso, come Aldo Favini e Silvano Zorzi.
Quando alla fine della guerra Colonnetti è nominato presidente del CNR, Franco Levi, sempre al suo fianco, coordinerà il Centro Studi sugli Stati di Coazioni Elastiche, una costola del CNR, dove, a stretto contatto con gli ingegneri sperimentatori, si dedica all'elaborazione delle teorie sulla viscosità, prendendo in considerazione il comportamento del precompresso nel tempo, le deformazioni lente e il ritiro del calcestruzzo, i fenomeni di rilassamento dei cavi, e così via.
In Italia le primissime sperimentazioni e i primi studi sono condotti, oltre che da Colonnetti, dall'ingegnere romano Carlo Cestelli Guidi (1906-1995), con la realizzazione di travi prefabbricate a fili aderenti.
Mentre Colonnetti ha soprattutto un ruolo di guida e di riferimento per lo sviluppo della teoria, tra accademici e sperimentatori, Cestelli Guidi si afferma come straordinario divulgatore presso gli ingegneri progettisti che intendono misurarsi con la costruzione in precompresso. Su questo fronte il suo ruolo risulta determinante sia attraverso le lezioni tenute fin dal 1944 alla facoltà di Ingegneria di Roma, che contemplano il calcolo e il progetto in precompresso, sia con la pubblicazione, nel 1947, del volume *Il conglomerato precompresso: teoria, esperienze, applicazioni.* Il testo, che si afferma immediatamente come un vero e proprio manuale della costruzione in precompresso, viene tradotto in più lingue e riedito ripetutamente, tanto che ancora oggi è facilmente reperibile. Lo stesso Cestelli Guidi, nella prefazione alla prima edizione, oltre a sottolineare che la materia è ancora in fase di studio e che solo dall'esperienza si conoscerà precisamente il comportamento di queste strutture una volta soggette ai carichi, premette la natura essenzialmente pratica del volume e rimanda, per gli aspetti teorici, alle "opere classiche" di Colonnetti.
In definitiva in Italia sono di primissimo livello gli studi teorici di Colonnetti, Levi e Arturo Danusso (1880-1968), mentre per la divulgazione della tecnica prima spicca fra tutte l'azione di Cestelli Guidi.
Quando alla fine del 1947 il Consiglio Superiore dei Lavori Pubblici, in stretta collaborazione con il CNR, emana le prime normative per la costruzione in cemento armato precompresso, incarica il ministero dei Lavori

Pubblici e lo stesso CNR di valutare la correttezza dei progetti delle opere in precompresso. Levi sarà l'ingegnere che si impegna personalmente a monitorare e verificare molte tra le più importanti opere in precompresso costruite in Italia. In tal modo i cantieri diventano un laboratorio formidabile per l'ingegnere, teorico del precompresso, dove verificare ipotesi e suggestioni. Quindi pratica sperimentale e teoria avanzano congiuntamente e procedono alla messa a punto di sistemi collaudati e scientificamente verificati.

L'immersione nella pratica costruttiva di Levi si completa quando progetta, in collaborazione, alcune opere impegnative come il Nuovo Palazzo delle Mostre, più noto come Palazzo a Vela, a Torino per l'Esposizione del 1961; la volta di copertura dello stadio di Teramo e il più grande bacino di carenaggio costruito a Genova, in seguito trasferito in Turchia

Il calcolo esatto per le opere in precompresso è difficile, non si conoscono bene i comportamenti di queste strutture che la sola teoria elastica non riesce a esplicitare: pertanto il cantiere diventa il momento delle prove e delle esperienze. Per sviluppare questo versante viene fondato nel 1951 l'Ismes (Istituto Sperimentale Modelli e Strutture, Bergamo), specificamente finalizzato alle prove su modelli.

Tuttavia non sarebbe corretto affermare che in Italia lo sviluppo del precompresso sia frutto della collaborazione fattiva di un gruppo di ingegneri, pratici e teorici; in effetti esso è piuttosto il risultato della convergenza di sforzi individuali: numerosi ingegneri, in modo autonomo e competitivo, hanno aggiunto il proprio tassello allo sviluppo e all'affermazione di questa tecnica costruttiva.

I dispositivi tecnici italiani non hanno la stessa innovativa originalità di quelli d'Oltralpe, eppure ogni ingegnere italiano che sia promotore e costruttore in precompresso, tenderà a brevettare e utilizzare il proprio sistema. Realizzando portali sperimentali e mettendo a punto sistemi di precompressione italiani, i vari Rinaldi, Balducchi, Morandi e Favini, li depositano come brevetti con privativa, anche se in genere si tratta piuttosto di ingegnose filiazioni e miglioramenti di sistemi francesi o tedeschi già collaudati.

Per meglio intendere questo delicato e interessante aspetto, è utile precisare che esistono due diversi sistemi di precompressione: uno, detto ad armatura pre-tesa, è fondato sulla messa in tensione delle armature prima di eseguire il getto delle travi cementizie all'interno di casseforme; una volta raggiunto l'indurimento delle travi, si rilasciano i cavi tenuti in tensione. Un secondo, più utilizzato e sperimentato, è prodotto attraver-

so il passaggio di cavi all'interno di guaine appositamente predisposte in sezioni cementizie già stagionate. Il tiro dei cavi in questo caso avviene a indurimento avvenuto, tramite l'uso di martinetti idraulici posti alle teste delle travi.
Le prime opere realizzate consistono in travi con sistemi statici di appoggio-appoggio e privilegiano la precompressione di post-tensione a cavi scorrevoli. Se i dispositivi non aggiungono innovazione agli esiti d'Oltralpe, i risultati costruttivi italiani riescono invece a competere e, in alcuni casi, a superare le opere realizzate in Europa. Gli anni precedenti il conflitto mondiale, con la scarsità di materie prime conseguenti all'autarchia, sono all'origine di una vicenda singolare dello sviluppo del precompresso in Italia, che probabilmente fornisce la chiave di lettura più perspicua per capire la storia costruttiva del cemento e del cemento armato precompresso in speciale modo.
Si deve agli innovativi studi di Tullia Iori sul cemento armato in Italia la limpida messa a fuoco di questa vicenda che si dispiega dalla nascita allo sviluppo del cemento armato, con particolare attenzione agli anni dell'autarchia. Quello che nasce come un severo limite per il mondo della costruzione in Italia diventa, nei fatti, spinta propulsiva per un'originalità di ricerca e una storia unica ed eccezionale.
I tecnici, costretti a interrompere cantieri e progetti di lavoro a causa degli eventi bellici, si dedicano a sperimentazioni sul cemento armato, perseguendo l'obbiettivo di ridurre al minimo l'uso del ferro, destinato quasi totalmente allo sforzo bellico.
In quegli anni tribolati sono decine i dispositivi brevettati per ridurre l'uso del ferro nelle opere cementizie: si oscilla da proposte di materiali alternativi, come il flessibile legno o le più esotiche canne di bambù, fin all'ipotesi di eliminare il ferro, sfruttando sistemi di incastro per la tenuta di laterizi e cemento. All'interno di questo mondo di sperimentazioni si inseriscono le ricerche sul cemento armato precompresso. Il precompresso sembra suggerire una via parallela e alternativa: infatti attraverso il suo uso si possono ridurre sensibilmente gli spessori del calcestruzzo e di conseguenza anche la quantità di armature.
Una strada diversa, che gli ingegneri italiani imboccano con grande entusiasmo e dedizione e che continueranno a perseguire anche dopo la guerra quando, con Cestelli Guidi presidente, nasce nel 1949 l'Associazione Nazionale Italiana Cemento Armato Precompresso, che riunisce gli ingegneri che studiano e sperimentano il precompresso.

Negli anni si avvicenderanno alla presidenza ingegneri del calibro di Luigi Stabilini e Aristide Giannelli.
Quando alla fine degli anni quaranta, dopo la realizzazione di piccole strutture dimostrative, le sperimentazioni approdano alla costruzione dei primi ponti, scatta una sorta di gara tra i progettisti per aggiudicarsi il primato della costruzione del primo ponte in precompresso.
Il primato dei ponti, sulla base della rassegna redatta dall'Aitec, spetterebbe a quello sul Piave a Vallesella (Belluno), costruito dall'ingegnere Carlo Predella tra il febbraio del 1949 e l'aprile del 1950: lungo 267 m, è impostato su sette campate e precompresso con sistema Freyssinet. Altre fonti indicano come primo il ponte costruito sul torrente Samoggia a Bologna da Giuseppe Rinaldi, tra l'ottobre del 1949 e il gennaio del 1950. Il ponte è a campata unica, in conci prefabbricati resi solidali mediante cavi post-tesi con dispositivo di precompressione Magnel; il sistema statico è quello di una trave appoggiata-appoggiata con una luce libera di 26 m.
Tra i numerosi sperimentatori italiani del precompresso emerge di gran lunga Riccardo Morandi non solo per la quantità di costruzioni, ma soprattutto per la qualità eccezionale delle opere, edificate utilizzando un sistema di precompressione da lui stesso brevettato. La permanenza dell'archivio dello studio Morandi e la sua disponibilità alla consultazione hanno consentito di ricostruire in modo dettagliato l'iter che ha portato dalle prime sperimentazioni ai progetti e alle straordinarie grandi opere della costruzione.
Per Morandi il precompresso non è solo una nuova tecnica costruttiva ma anche una sistema per consolidare strutture murarie antiche, come quelle romane dell'Arena di Verona. Fin dagli anni trenta del novecento si teme per la stabilità di un settore dell'Arena veronese che presenta uno strapiombo verso l'esterno e risulta scollegata dal resto della struttura muraria. Durante il secondo conflitto mondiale, nel timore che si potesse verificare un crollo a seguito di bombardamenti o anche delle sole vibrazioni conseguenti alle esplosioni, sono costruiti in gran fretta provvisori speroni murari di sostegno. Nel 1954 Morandi progetta e realizza l'innovativo progetto di rafforzamento della struttura muraria tramite l'inserimento e l'ancoraggio, a terra e in sommità, di cavi di precompressione; collegando con i cavi anche trasversalmente oltre che verticalmente, le murature, si poterono demolire in sicurezza le arcate provvisorie, conservando intatte le strutture murarie.
Naturalmente il precompresso è adottato da Morandi anche e soprattutto come tecnica costruttiva e di montaggio: si veda l'ardita passerella pedo-

nale a Vagli di Sotto, in Alta Garfagnana, sempre nel 1954, dove le due semi-arcate che sostengono la campata sono costruite rispettivamente sui due fianchi e poi sono fatte ruotare fino a incontrarsi nel punto di colmo, attraverso delicate e sempre variabili tesature dei cavi di precompressione delle due travi a semiarco.
Non solo per Morandi, il precompresso è infine ricorrente nella costruzione di serbatoi; di coperture a travetti prefabbricati: in definitiva diventa un modo di costruire che rivoluziona le condizioni del progetto e di applicazione del calcestruzzo semplice, a cui spesso si associa.
Nell'Esposizione di Torino del 1961 la tecnologia di avanguardia si materializza ostensivamente nello scattante percorso aereo a quota 6 m che, progettato da Morandi in cemento armato precompresso, supportava la monorotaia Alweg, su cui correva in convoglio che congiungeva tutta l'area espositiva.

Diffusione e sviluppo del precompresso

Il precompresso continua ad essere impiegato nelle grandi opere e gli ingegneri che in Italia ne furono i pionieri rimasero sostanzialmente fedeli a questo sistema costruttivo. A partire dagli anni settanta in poi sarà non più Morandi in Italia ad egemonizzare le grandi opere in precompresso, ma Silvano Zorzi, che costruirà numerosissime opere, soprattutto ponti e viadotti. Zorzi, allievo di Colonnetti in Svizzera, emerge per estro e produttività tra i protagonisti della costruzione in precompresso. Nel 1961 fonda la società In.Co., Ingegneri Consulenti, uno tra i pochi se non l'unico studio di ingegneria della stagione eroica del Novecento ancora in attività, grazie all'azione dell'ingegnere Aldo Müller, collaboratore di Zorzi fin dagli anni sessanta. Come emerge dai fondamentali studi pubblicati dall'Aitec, che compendiano esaustive rassegne delle grandi opere italiane in precompresso, dagli anni '70 in Italia prevalgono le opere di Zorzi: sono soprattutto ponti e viadotti, ma anche opere idrauliche, grandi serbatoi, pavimentazioni stradali, silos e perfino architetture civili.
Nel libro sono poi analizzate in modo approfondito dieci costruzioni in precompresso, realizzate in Italia tra il 1950 e i tempi più recenti, che testimoniano più di cinquant'anni di evoluzione della tecnica, del suo impiego e delle sue potenzialità architettoniche.
Tra le numerose opere costruite in Italia in precompresso a partire dal dopoguerra, in questo scritto si sono privilegiate quelle più straordinarie e innovative, non solo per la soluzione tecnica adottata, ma anche per la persuasività architettonica e l'estro formale che le distinguono.

Allo stato attuale l'uso del precompresso, pur se ancora molto diffuso, è in genere circoscritto alle opere infrastrutturali: grandi impalcati stradali per ponti, grandi luci e opere straordinarie per impegno tecnico, mentre è decisamente scemata l'adozione in opere civili più piccole. Molto spesso sono impiegati elementi prefabbricati in precompresso per le coperture di capannoni industriali, mentre il precompresso gettato in opera è ascritto solo alle grandi opere.

Progetti in cantiere

Il primo caso studio è quello del cantiere, ora concluso, della Biblioteca Hertziana di Roma, dove la necessità di salvaguardare un'eccezionale stratificazione archeologica sotterranea, è stata risolta grazie a colossali travi in cemento armato precompresso.
Nel 1996 è designato vincitore del concorso internazionale per l'ampliamento della Biblioteca Hertziana, nel cuore di Roma, lo spagnolo Juan Navarro Baldeweg (Parducci, 2009). Il progetto interessa l'adeguamento dell'edificio che ospita la biblioteca fondata nel 1913 per promuovere la ricerca e lo studio in Germania della storia dell'arte italiana. Oggi l'Hertziana è tra le biblioteche specializzate in arte più importanti del mondo. Nella centralissima zona di Trinità dei Monti a Roma, nel lotto triangolare stretto tra via Gregoriana e via Sistina, l'intervento si concentra sul palazzo Nuovo, interposto tra gli storici palazzetti Zuccari e Stroganoff, anch'essi di proprietà della biblioteca tedesca. Il progetto di ristrutturazione di Baldeweg mantiene i due fronti su strada del palazzo preesistente, ne demolisce integralmente le partizioni interne, risalenti agli anni sessanta, per creare ambienti idonei alla nuova biblioteca. Quattro piani fuori terra, con ballatoi per le sale letture e il deposito libri, si sviluppano intorno ad una corte interna, e si innalzano su un basamento interrato di nove metri che alloga i depositi librari. La presenza dei resti della villa di Lucullo di epoca repubblicana nel piano di scavo delle fondazioni e la volontà di proseguire lo scavo archeologico anche durante i lavori in elevazione, costringono i progettisti delle strutture, lo studio Tekno IN ingegneri associati di Roma di Alberto Parducci, Alfredo Marimpietri e Marco Mezzi, all'adozione di un'originale soluzione tecnica. Su una doppia fila di pali di fondazione, disposti in corrispondenza dei marciapiedi, lungo le due strade limitrofe, sono poggiate due massicce travi di cemento armato al di sopra delle quali si situa il piano-trave in cemento armato precompresso gettato in opera. Le travi in precompresso si attestano così come vere e proprie pareti perimetrali

e interne del piano a quota -3 m: in esse sono ritagliate le aperture per l'accesso ai vani ricavati nel basamento strutturale, costruito come fosse un ponte ipogeo, sospeso sulle aree di scavo archeologico. La struttura a travi in precompresso occupa un intero piano che, alto circa tre m, profila una maglia scatolare, totalmente accessibile, che sostiene l'intero edificio: essa supporta i telai metallici superiori e regge i piani interrati, che sono completamente appesi ad essa, così da non intaccare l'area archeologica sottostante. Una struttura a ponte in precompresso sostiene l'intero edificio e configura la maggiore difficoltà cantieristica dell'edificio, ma anche la soluzione più originale adottata. L'uso del precompresso nell'Hertziana evitando strutture di fondazione all'interno del sedime archeologico, consente di approfondire gli scavi anche durante il cantiere, che si è poi concluso nel 2013.
Anche in sofisticate e raffinate opere con struttura portante in normale cemento armato, come il centro civico polivalente recentemente costruito dall'architetto Emanuele Fidone a Modica, si ricorre a travi di copertura in precompresso, necessarie a coprire ampie luci libere e a consentire lunghissime asole di affaccio. Situato in una zona periferica, il centro civico, un manufatto basso e allungato, dall'aspetto ermetico ed elegante, occupa un lotto trapezoidale ed è formato da una lunga galleria con i servizi, gli uffici e le piccole sale che conduce alla sala polivalente, secondo una impostazione planimetrica a bandiera. Anche in un piccolo edificio come questo l'architetto ha preferito l'uso di travi prefabbricate in precompresso, poiché consentono un risparmio di denaro e di tempo, superando l'audace luce di ben 20 m della copertura della sala. Se nella grande sala un controsoffitto dissimula le travi alla vista, una grande trave grezza è stata lasciata in vista delle rampe che circondano la sala.
Un'altra opera interessante è il Centro Regionale dell'Umbria della Protezione Civile di Foligno; programmato dal 1998 e iniziato nel 2008 nella zona nord di ingresso alla città. Il complesso, composto da più edifici, progettati da Alberto Parducci su incarico della Regione Umbria, occupa un'area di 7800 mq dove sono disposti: un edificio per "Emergenza e Formazione", una sala operativa per la Protezione Civile e una palazzina per gli impianti del Centro. L'applicazione più significativa del cemento precompresso è quella della Centrale Operativa che, progettata in collaborazione con l'architetto Guido Tommesani, consiste nei tre piani di una falsa cupola di 32 metri di diametro. La configurazione strutturale fornisce l'esempio di una morfologia architettonica finalizzata all'applicazione ottimale delle tecniche di isolamento sismico.

L'edificio poggia soltanto su dieci isolatori disposti lungo il perimetro di base, capaci di sopportare, senza danno, spostamenti orizzontali di 405 mm. Un sistema di vele di cemento armato sostiene il solaio del primo piano, dal quale sorgono dieci archi, anch'essi di cemento armato. Alla chiave degli archi è appeso un nucleo centrale, precompresso mediante cavi Dywidag, nel quale sono inseriti i percorsi verticali che costituiscono il sostegno interno dei solai superiori. La costruzione è stata progettata per resistere a un terremoto di una PGA (Peak ground acceleration, misura dell'accelerazione del terremoto) di 0.49g, previsto in sito con periodo di ritorno di 1000 anni. La configurazione strutturale, molto compatta, è tale da consentire all'edificio, in presenza dello sciagurato evento, di oscillare lentamente con un periodo di 2.65 secondi, mantenendosi praticamente indeformato (progetto vincitore del *Premio AICAP 2011*, categoria edifici e dell'*European Concrete Award 2012*).
Alcuni progetti innovativi e sperimentali impiegano anche il precompresso: è questo il caso dell'Innovation and Technology Central Laboratory (ITC Lab) di Bergamo, il nevralgico centro Italcementi per la ricerca e l'innovazione, laboratorio di riferimento per la sperimentazione e la diffusione del cemento in Italia, il cui edificio è a sua volta oggetto di sperimentazione. Esso si inserisce tra gli edifici del masterplan (2003) elaborato dall'architetto francese Jean Nouvel per il cosiddetto Kilometro Rosso, il Parco Scientifico e Tecnologico che si snoda lungo l'autostrada Milano-Venezia. Dopo la feconda collaborazione attuata per la costruzione della chiesa di Tor Tre Teste a Roma, l'Italcementi si rivolge di nuovo nel 2005 all'architetto americano Richard Meier per il progetto del complesso per laboratori, uffici, biblioteca e spazi sociali. Su un'area di 11.000 mq l'architetto disegna una pianta triangolare, con 2 piani interrati e 2 piani fuori terra, i cui lati sono dettati dall'autostrada e dalla via interna. I laboratori, aperti su un patio, sono schermati verso l'autostrada da setti grigliati; nel fronte lungo la strada interna, un muro cieco e una vetrata, che scherma una doppia rampa, delimitano gli uffici e gli ambienti che accoglieranno la più grande biblioteca italiana sul cemento. Sistemi innovativi e diversi tipi di lavorazione e messa in opera del cemento scandiscono la costruzione dell'edificio, la cui copertura è in parte realizzata in cemento armato precompresso.
Alcune di queste opere sono indagate nel volume, e i casi approfonditi perseguono il semplice scopo di testimoniare la vitalità e l'attualità di una tecnica costruttiva che ha contrassegnato la costruzione italiana dell'ultimo mezzo secolo.

Questo mio approccio alla ricerca, che affronta con i metodi della storia dell'architettura, il tema della storia e costruzione degli edifici, in particolare applicato alle grandi opere di ingegneria, ha rappresentato un mio carattere di originalità, che mi ha permesso di diventare un ricercatore affermato. Sono infatti stata invitata a presentare le mie ricerche in tantissime università. A cominciare dal prestigioso invito della prof. Alina Payne, titolare della cattedra di Storia dell'Arte e dell'Architettura alla Harvard University, Cambridge Massachusetts, a tenere un seminario, il 1° marzo 2011, dal titolo *Engineers build Rome: Pier Luigi Nervi, Riccardo Morandi and the1960 Olympic Game* al Department of the History of Art and Architecture. Con una tavola rotonda composta dai più importanti docenti del dipartimento: James Ackerman, Michael Hays, Neil Levine, Erika Naginsky e Antoine Picon.
Segnalo inoltre la conferenza tenuta il 1° maggio 2013 all'École Polytechnique Fédérale De Lausanne (EPFL-Losanna) nel ciclo di conferenze "Tribute to the history of Engineering", organizzate in concomitanza con la mostra itinerante *Pier Luigi Nervi. Architettura come sfida*, a cura di C. Olmo, sono invitata a presentare le mie ricerche in un incontro seminariale a due voci, con la prof. Pepa Cassinello, direttrice della Fondazione Eduardo Torroja di Madrid.
A distanza di anni, sarà proprio la direttrice Cassinello a invitarmi, 21 novembre 2018, a presentare la ricerca dal titolo *Riccardo Morandi (1902-1989) Italian Engineer: "Le Corbusier on four wheels" (By Bruno Zevi)* all'International Conference on Construction Research Eduardo Torroja. Architecture, Ingeneering and Concrete – AEC (Madrid, Fundacion Eduardo Torroja, 21-22-23 November 2018), invitandomi a ricoprire anche il ruolo di Membro del Comitato Scientifico del Convegno.
Su questi stessi temi, tra architettura e costruzione, che sono divenuti centrali nelle mie ricerche avviate a partire dal volume presentato e premiato, sono stata intervistata dalle più importanti testate giornalistiche internazionali all'indomani della tragedia del crollo del viadotto del Polcevera, cito il più prestigioso tra tutti: "The New York Times", 6 settembre 2018.
I miei studi sull'ingegneria e in particolare su Morandi mi hanno portato poi a collaborare con l'amministrazione comunale di Colleferro, una città a sud di Roma, un centro urbano progettato negli anni trenta dal giovane Morandi. Grazie alla vincita del premio *Città della Cultura della Regione Lazio 2018*, l'amministrazione di Colleferro ha attuato un piano di promozione e valorizzazione dell'architettura di quella che è stata definita la Città Morandiana, raccontata nel film *Città Novecento* (2021), selezionato alla

festa del Cinema di Roma e uscito nei cinema. Ho partecipato alla costruzione del progetto come membro del comitato scientifico e sono uno dei protagonisti intervistati nel film.

Proprio il mio profilo di storico dell'architettura con una formazione e interessi volti alla storia della costruzione mi hanno portato a vincere lo scorso marzo 2021 il concorso da professore associato all'università IUAV di Venezia, dove sto collaborando alla costituzione del gruppo di ricerca sull'Arte del Costruire, oltre ad aver pubblicato, con il prof. Marko Pogacnik, un volume dedicato all'Ingegneria Italiana come fattore di affermazione del *Made in Italy* nel Novecento (Marandola, Pogacnik, 2022).
Ancora nelle mie più recenti ricerche, spiccano come un tassello distintivo e fondamentale i pionieristici studi sulla costruzione in cemento armato precompresso e le opere italiane costruite con questo sistema. La pubblicazione del libro, risultato di lunghe e innovative ricerche, e menzione speciale al premio Internazionale Edoardo Benvenuto ha rappresentato un passo importante della mia carriera, forse il più originale risultato, e il premio è stato per me un importante pungolo che mi ha spinto a proseguire in questa direzione, seguendo un percorso inedito e originale nel campo della storia dell'architettura.

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Fig. 1. M. Marandola, *La costruzione in precompresso. Conoscere per recuperare il patrimonio italiano*, Milano, Il Sole24ORE, 2009, copertina.

Fig. 2. R. Morandi, passerella pedonale a Vagli di Sotto, Lucca, 1952-1954.

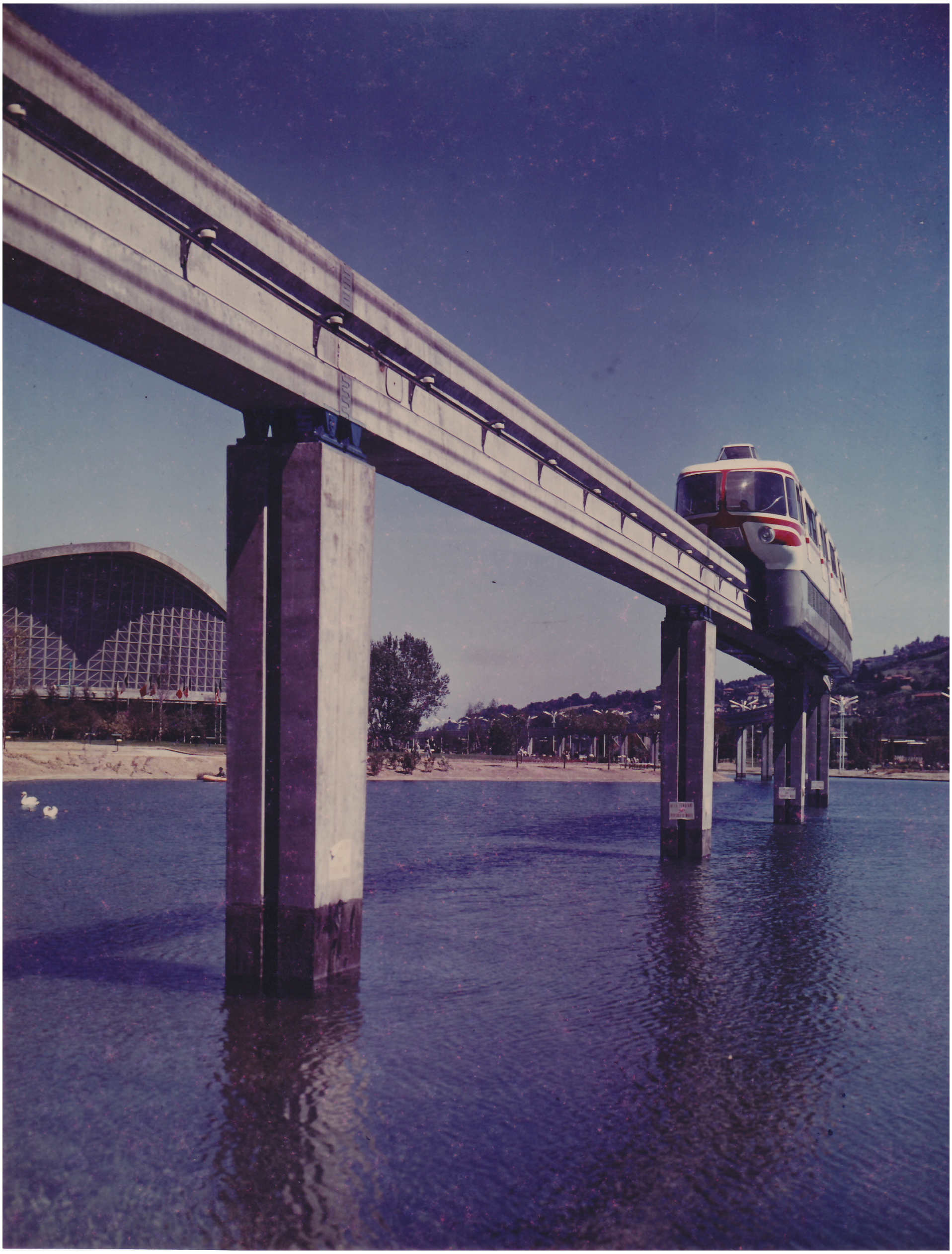

Fig. 3. R. Morandi, monorotaia Alweg per il collegamento dell'area di Torino Esposizioni, 1961.

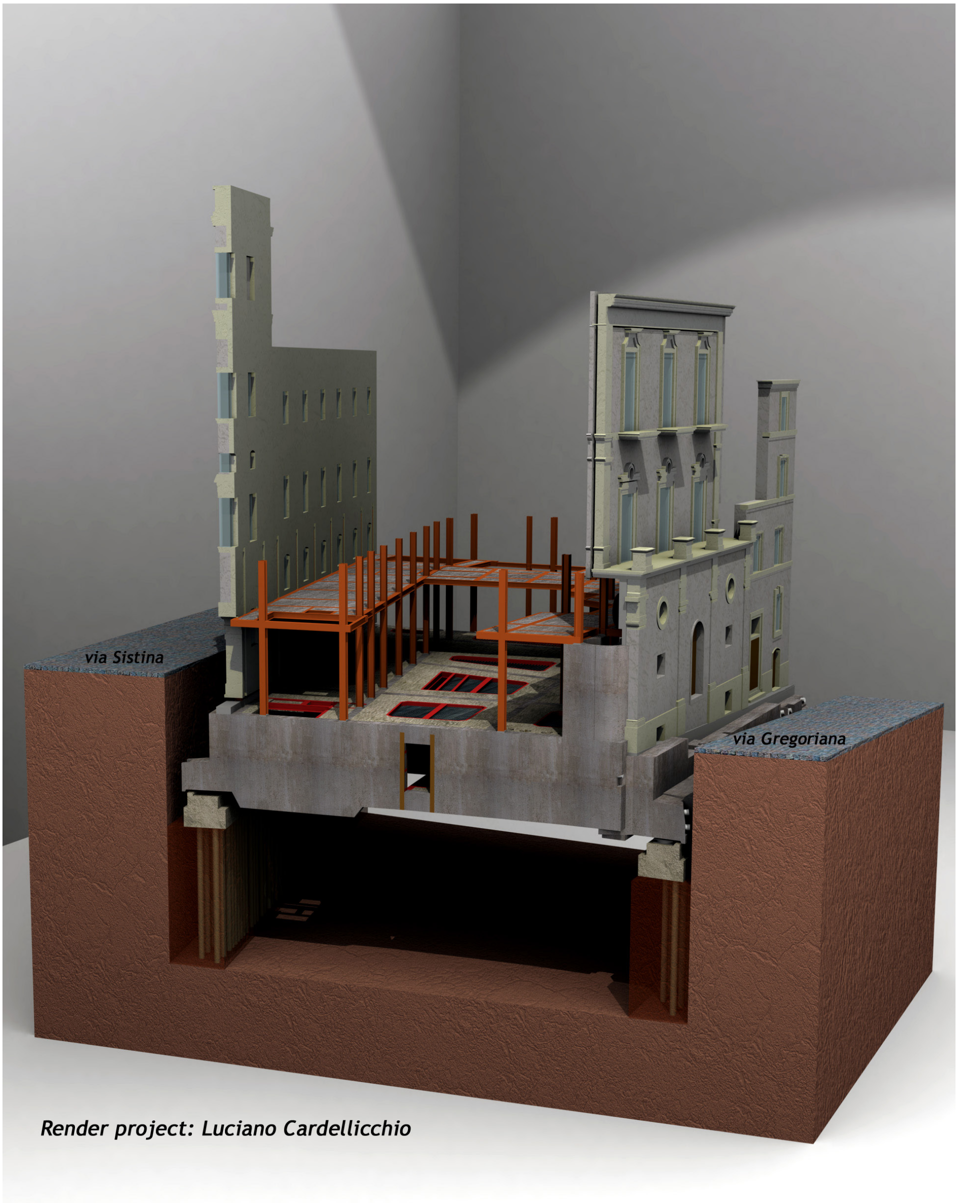

Fig. 4. Juan Navarro Baldeweg, progetto della nuova biblioteca Hertziana di Roma. Rendering costruttivo dell'ing. Luciano Cardellicchio.

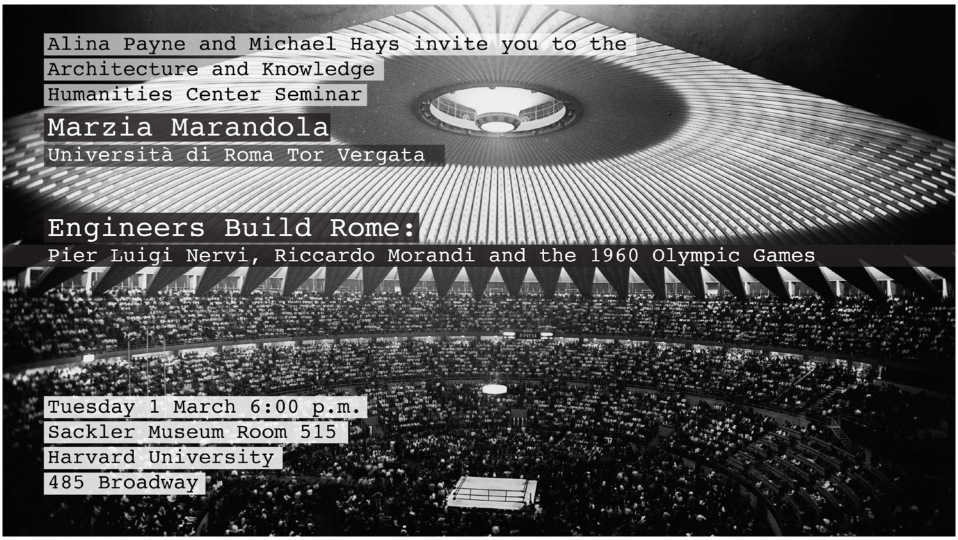

Fig. 5. Locandina della conferenza presso la Harvard University, 2011.

Fig. 6. Locandina del film *Città del Novecento,* 2021.

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Advances in Thrust Network Analysis. Constrained equilibrium assessment of masonry vaulted structures

1. *Introduction*

The assessment of unreinforced masonry structures requires specialised tools that are not widely available (Tralli, Alessandri, Milani, 2014). Structural modelling of masonry should take into account the unilateral behaviour of the material, related to its high compressive and low tensile strength. Besides, the collapse of masonry structures is usually a result of lack of stability rather than insufficient material strength (Huerta, 2001; Ochsendorf, 2002). For that reason, general-purpose analysis tools which focus on computing the current internal stresses can not be directly applied to the masonry without detailed considerations (Shin *et al*., 2016).
As an alternative, Heyman (1966, 1995) showed that Limit Analysis can be applied to masonry structures given that three assumptions are respected: the material has no tensile strength, infinite compressive strength, and sliding does not occur. Within this framework, the collapse or limit states are searched. In particular, the safe, or lower-bound theorem of limit analysis applies. The theorem states that if at least one admissible stress state can be found, the structure is safe in its current state. When applying the safe theorem to masonry, admissible stress states correspond to compression-only force paths or thrust lines in equilibrium with the applied loads and contained within the geometry of the structure, i.e., between intra- and extrados.
The search for compressive, internal force paths in masonry has been historically applied to arches (Moseley, 1843), sliced domes (Poleni, 1748) and vaults (Ungewitter, 1890), and commonly applied in combination with graphic statics (e.g., Wolfe, 1921). Since these techniques are justified by the safe theorem, they result in a lower-bound (and safe) estimation of the actual capacity of the structure. Currently, this technique is known as Thrust Line Analysis (TLA) and it remains a contemporary tool for the assessment of two-dimensional, or sliced 3D structures (Smars, 2000; Zessin, Lau, Ochsendorf, 2010; Angelillo, Olivieri, DeJong, 2021) and the core of masonry analysis tools (e.g., Limit State: RING, 2021).

Given the independence of major mechanical parameters, lower-bound methods are an important tool for the structural assessment of historic structures, relying exclusively on geometrical data from the intrados and the extrados (Huerta, 2001).
Nevertheless, the extension of lower-bound methods to three dimensions is challenging due to the increased degree of indeterminacy of the three-dimensional equilibrium. The slicing technique (Huerta, 2008) can not explore fully three-dimensional force flows that arise in spatial masonry structures. In recent years, various lower-bound equilibrium formulations have been developed to cope with the assessment of three-dimensional masonry structures. These strategies can be divided into continuous and discrete approaches.
Continuous approaches search for compressive membranes within the geometry of the structure. Heyman (1977) shows that purely applying shell analysis (Calladine, 1983) taking the middle surface of masonry structures often results in tensile membrane stresses, which require an adaptation of the methodology by, e.g., considering sections or slices. Another description of this problem, based on continuum mechanics, takes as variable both: the internal stresses of the membranes and their geometry. The equilibrium equations are solved by assuming a Pucher formulation and considering the potential stress (or Airy) functions to describe the internal distribution of the stresses (Fraternali et al. 2002). With such an approach, the vertical equilibrium is described by a second-order differential equation. To solve the differential equation, different approaches have been developed. In Fraternali (2010) and Angelillo, Babilio, Fortunato (2013), an approximation of the relevant functions is done in polyhedral domains; in Fraddosio, Lepore, Piccioni (2020), a polygonal approximation is used in an equally spaced point grid; in Miki, Igarashi, Block (2015) NURBS surfaces are used and in Baratta, Corbi (2010) analytical solutions are obtained for simple geometries.
On the other hand, discrete methods can be applied avoiding the computation of differential equations offering a flexible formulation that allows considering a wider range of discontinuities in loading, geometry, and boundary conditions. O'Dwyer (1999) proposed a method considering the internal forces in the masonry as a network carrying only compressive, axial forces. Different load cases and flow of forces can be quickly investigated, and the final solution allows to verify and modify the equilibrium by directly changing the internal axial forces at the edges of the network. Nevertheless, in O'Dwyer (1999), no strategy is applied to deal with the indeterminacy of the projected layout of forces.

Following the work of O'Dwyer (1999), Block and Ochsendorf (2007) formalised Thrust Network Analysis (TNA) as a method to compute the internal forces as a network of thrusts based on graphic statics. With such an approach, the spatial equilibrium of networks can be executed interactively by connecting the magnitude of the internal forces with a force diagram analogous to the force polygons in graphic statics (Block, 2009). Further developments of TNA included the framing of the problem as a constrained application of the Force Density Method (Schek, 1973); the introduction of the concept of independent edges, which allow for efficiently searching equilibrium states for thrust networks with a fixed projection (Block, Lachuer, 2014; Van Mele, Block, 2014); and, allowing this equilibrium exploration as a search of admissible solutions through optimisation processes (Block, 2009; Van Mele *et al*., 2014; Block, Lachauer, 2014). Recent developments extended this framework to determine the level of stability of three-dimensional masonry structures based on the sequential solving of a series of optimisation problems with different objective functions (Maia Avelino *et al*., 2021) and to consider internal states arising for prescribed foundation settlements (Maia Avelino *et al*., 2022).
This paper gives an overview of the major concepts behind Thrust Network Analysis in the context of masonry assessment. It summarises the recent advances in the method that allow it to be used to search for admissible stress states in three-dimensional masonry structures with generic geometry and output relevant feedback about the level of stability of the structure.
The paper is organised as follows: Section 2 presents the graphic statics based formulation of TNA. Section 3 presents a numerical formulation of TNA suitable for constrained optimisation. Section 4 formulates constrained nonlinear optimisation problems with TNA that are relevant to assess masonry structures. In Section 5, applications to three-dimensional masonry structures are presented. Finally, in Section 6, the conclusion and outlook of the method are described.

2. *Graphic statics based approach*

This section presents a graphic statics based approach to TNA (2.1) that is analogous to the equilibrium problems in graphic statics. Following, an iterative algorithm to solve the equilibrium in this framework is formulated (2.2).

2.1. *On the equilibrium of projected networks*

Graphic Statics is a well-known method to find the equilibrium of two-dimensional structures. The relationship between the structure's geometry and its internal forces is described by the reciprocal relation between two diagrams (Culmann, 1875; Wolfe, 1921), the form and force diagrams. The former describes the geometrical configuration of the (axial) internal forces, and the latter represents their equilibrium. A closed polygon in the force diagram represents the equilibrium of a node in the form diagram. Graphic statics offers an intuitive evaluation of the structural equilibrium. It was used as a main structural calculation tool in the late 19th and early 20th century. For masonry analysis, graphic statics offers a straightforward way to find a thrust line within the geometry of a structure. **Figure 1** shows an admissible compressive equilibrium solution, or thrust line (G), found for the semi-circular arch (Λ). The equilibrium in each node of the thrust line relates to the equilibrium of a portion (e.g., a block) subjected to its weight w_i. The force diagram (Γ^*) represents the sum of all closed polygons representing the local equilibria in the nodes of the thrust line. In the force diagram, the length of each segment relates to the force carried in the corresponding segment in the thrust line. By modifying the force diagram, e.g., by moving the pole point (o) and choosing the coordinates of one of the nodes of the form diagram, the internal (and reaction) forces and the shape of the thrust line can be changed, and different equilibrium solutions explored.

The graphic formulation of TNA offers a 2.5D extension of 2D graphic statics (Block, Ochsendorf, 2007). In this formulation, the loading case is assumed to be parallel, which often occurs in masonry structures, e.g., the gravity loads and horizontal force multipliers. When all loads are parallel, the spatial thrust network can be projected onto a plane perpendicular to the loads resulting in a two-dimensional graphic statics problem in which the applied loads vanish.

For the gravity loading case, the projection of the thrust network (G) results in the planar form diagram (Γ) for which the horizontal equilibrium can be described graphically with the construction of the force diagram (Γ^*), respecting the same well-known two-dimensional graphic statics procedures. In this case, Γ^* represents the equilibrium of the horizontal components of the forces, i.e., the (horizontal) thrusts of G. If we formalise these concepts in TNA, the horizontal equilibrium is verified when the diagrams are reciprocal, which implies that:

a. All corresponding edges in the form and force diagrams are parallel.
b. The length of an edge in the force diagram is proportional to the axial force, carried by its corresponding edge in the form diagram.
c. Each node in the form diagram is represented by a closed polygon in the force diagram.

Figure 2 shows a thrust network (G) whose horizontal equilibrium can be described graphically by their reciprocal form and the force diagrams. This thrust network models a possible distribution of the internal forces in a masonry cap (Λ), i.e., each vertically applied load relates to the weight of a portion (e.g., a block) of the structure and the position of the vertices match the vertical projection of the centroids. The thrust network (G) is admissible as all forces are compressive and the network fits within the geometry of Λ. As in the 2D problem, modifying the geometry of the force diagram while respecting reciprocity with the form diagram leads to different equilibrium solutions for that form diagram.
The form diagram represents the horizontal layout of the thrusts, i.e., and a map of the force flow within the structure. In the context of masonry assessment, the form diagram is chosen (or generated) by the engineer based on intuition or experience on likely force paths within the structure, usually following its main geometric features, such as principal curvature or creases.
With this approach, the equilibrium of a node in G can be divided into horizontal and vertical components. For node *i*, shown in **figure 3**, the horizontal and vertical equilibria are described by the following equations:

$$\begin{aligned} f_{ji}^H + f_{ki}^H + f_{li}^H &= 0, \\ f_{ji}^V + f_{ki}^V + f_{li}^V &= p_i, \end{aligned} \qquad [1.1\text{-}1.2]$$

where f_{ji}^H and f_{ji}^V describe the horizontal and vertical components of an edge's (axial) force connecting nodes *j* and *i*, and p_i is the load lumped in the node.
With the decoupling of horizontal and vertical equilibrium, there are multiple ways in which these equations can be treated and solved. In the following section, we discuss an interactive, graphic way to solve the horizontal equilibrium in Equation 1.1.

2.2. *Interactive graphic equilibrium*

We introduce the following nomenclature, the form diagram Γ is composed of *m* edges and *n* nodes with planar coordinates stored in the vectors **x**, **y**. Each edge $\mathbf{e}_{ij}$ connects vertices *i* and *j* with a length l_{ij} and force f_{ij}. The elements of the force diagram Γ* are marked by an asterix (*), such that the diagram has *m** edges and *n** nodes. Each edge $\mathbf{e}_{ij}$* of the force diagram has length l_{ij}*. The thrust network (G) corresponds to the vertical lift of Γ, where the vertical nodal coordinates are stored in the vector **z**.
To find reciprocal form and force diagrams, a parallelisation algorithm has been proposed in Rippmann, Lachauer, Block (2012) and is described herein. It starts with a form diagram and its dual diagram, so not yet a force diagram representing a possible equilibrium for it, and updates the vertex positions of both diagrams to match the target orientation of its corresponding edges, such that at the end of the process the diagrams are reciprocal. To obtain a force diagram, the algorithm starts from the centroidal dual topology of the form diagram. Details about the automatic generation of the centroidal dual topology are given in Rippmann, Lachauer, Block (2012).
To find an initial state of equilibrium, target vectors $\mathbf{t}_{ij}$ are imposed to the edges of the form and force diagrams to parallelise the corresponding edges. Not only the force diagram can be aligned with the original form diagram but a weighting factor γ = {0, ...,1} can be introduced, which increases or decreases the influence of the form diagram to define the target directions to be used in the parallelisation. The target vector $\mathbf{t}_{ij}$ for each pair of corresponding edges $\mathbf{e}_{ij}$ and $\mathbf{e}_{ij}$* is then computed as follows:

$$t_{ij} = \gamma * \hat{\mathbf{e}}_{ij} + (1-\gamma)\, \hat{\mathbf{e}}^*_{ij}, \text{ with } 0 \le \gamma \le 1, \qquad [2]$$

where $\hat{\mathbf{e}}_{ij}$ and $\hat{\mathbf{e}}_{ij}$* are the normalised directions of edges $\mathbf{e}_{ij}$ and $\mathbf{e}_{ij}$*.
With the target direction and vectors, the solver iterates over all nodes of the form and force diagrams and calculates an updated vertex positions $\mathbf{P}_i$ and/or $\mathbf{P}_i$*. The procedure is applied analogously to both diagrams and is described here to update the force diagram.
Let E_i represent the group of edges $\mathbf{e}_{ij}$* connected to a vertex $\mathbf{v}_i$* of the force diagram with target vector $\mathbf{t}_{ij}$ computed for a given Γ, and target length l_{ij}*. The updated position $\mathbf{P}_i$* of this node is computed by:

$$\mathbf{P}^*_{\mathrm{i}} = \frac{\sum_{j \in E_i}\left(\mathbf{v}^*_i + l^*_{ij} * \hat{\boldsymbol{t}}^*_{ij}\right)}{\mathrm{n}(E_i)}, \qquad [3]$$

where n(E_i) is the number of neighbors of vertex $\mathbf{v}_i$*.
This iterative approach is applied to all nodes of the form and force diagrams until the stopping criteria is reached, i.e. corresponding edges have the same direction within a chosen maximum deviation angle α_{MAX}. This interactive approach also indirectly imposes that the networks are subjected to compressive-only forces as for these cases Γ* must be composed of non-overlapping, convex spaces (Rippmann, Lachauer, Block, 2012).
With this algorithm, interactively exploring equilibrium interactively is possible by stretching or moving nodes in the force diagram, after which the iterative solver finds the new reciprocal state as close as possible to the modification considering the weighting factor γ.
Once horizontal equilibrium is achieved, the horizontal forces (f_{ji}^H) in the form diagram are taken as the lengths of the force diagram (l_{ji}^*) multiplied by a scaling factor 1/*r*:

$$f_{ji}^H = \frac{1}{r} * l_{ji}^* . \qquad [4]$$

Therefore, by stretching members of the force diagram, the force lengths l_{ij}* are increased and so are the forces in the corresponding edges of the form diagram f_{ji}^H. **Figure 4** shows four different equilibria obtained by re-distributing the horizontal thrusts in an orthogonal grid supported along its boundary. An equally distributed configuration is shown in **figure 4a**. In **figures 4b-d**, the forces are increased in the edges highlighted, attracting forces that result in shallow arches, or creases in the thrust networks (G).
Following the computation of a possible horizontal equilibrium through parallelisation, the vertical equilibrium is solved as a separate step. It corresponds to performing a lifting of the horizontally equilibrated network. Assuming the obtained horizontal forces (Equation 5), the vertical equilibrium (1.2) can be rewritten as:

$$f_{ji}^H * \frac{(z_i - z_j)}{l_{ij}^H} + f_{ki}^H * \frac{(z_i - z_k)}{l_{kj}^H} + f_{li}^H * \frac{(z_i - z_l)}{l_{li}^H} = p_i , \qquad [5]$$

which, after substituting the horizontal forces (4), can be rearranged as:

$$\left(\frac{l_{ji}^*}{l_{ji}^H} + \frac{l_{ki}^*}{l_{ki}^H} + \frac{l_{li}^*}{l_{li}^H}\right) z_i - \left(\frac{l_{ji}^*}{l_{ji}^H}\right) z_j - \left(\frac{l_{ki}^*}{l_{ki}^H}\right) z_k - \left(\frac{l_{li}^*}{l_{li}^H}\right) z_l = p_i * r , \qquad [6]$$

which is a linear function in terms of the height of the nodes (z_i) and the scaling factor. This scaling factor allows for searching for globally deeper or shallower thrust networks for a given horizontal equilibrium distribution, as shown in **figure 5**.
This framework has been applied to the design of new compressive vaulted structures. Manually stretching and moving nodes in the force diagrams bring intuitive and direct visual feedback to the horizontal distribution of forces and resulting thrust networks.
To make this intuitive forward TNA-based form-finding approach available, RhinoVault, a plug-in for the CAD software Rhinoceros, was developed (Rippmann, Lachauer, Block, 2012; Rippmann, 2016). Also, a Python-based code implementation of TNA has been made available as the *compas_tna* package (Van Mele, 2020), built on the open-source computational framework COMPAS (Van Mele *et al.*, 2017). Recently, RhinoVault 2, an implementation of the TNA package as a robust updated version of RhinoVault, has been released (Block Research Group, 2020).
However, when it comes to masonry assessment, the search for admissible networks needs to be automated since it relates to finding particular solutions that are contained within (usually tight) upper and lower geometric bounds. Manual manipulation of the diagrams to achieve such particular geometric solutions is tedious, if not impossible, so this search must be executed by means of optimisation. To cope with this automation, a numerical formulation of TNA is proposed based on Block (2019) and Block, Lachauer (2014), which will be described in the following section.

3. *Numerical TNA formulation*

A more robust numerical control of the equilibrium can be achieved by relating the use of form and force diagrams in TNA to the equilibrium of general networks through the use of the force density method (FDM) (Schek, 1974). This is shown in detail in Block (2009) and Block, Lachauer (2014) and will be summarised in Section 3.1. More specifically, TNA can be seen as a special case of FDM in which the horizontal projection of the equilibrium solutions, i.e. the form diagram, is constrained to remain fixed.
To restrict the problem to a fixed planar diagram, the degrees of freedom in the pattern must be identified, adding constraints to the full set of force densities, as described in Section 3.2.

3.1. *Constrained force density formulation*

To rewrite Equation 6 in matrix format, we introduce the connectivity matrix **C** [*m*x*n*], which describes the connectivity of an oriented network:

$$\mathbf{C} = \begin{cases} 1 & \text{if vertex } i \text{ is the head of edge } j \\ -1 & \text{if vertex } i \text{ is the head of edge } j \\ 0 & \text{otherwise} \end{cases} \qquad [7]$$

As shown in Equation 6, considering the quotient of the force and the length of the edge in the network linearises the vertical nodal position of the networks. This quotient is also known as the force density of an edge q_{ij}, defined and connected to the initial variables as follows:

$$q_{ij} = \frac{f_{ij}}{l_{ij}} = \frac{f_{ij}^H}{l_{ij}^H} = \frac{1}{r} * \frac{l_{ij}^{H\,*}}{l_{ij}^H} \qquad [8]$$

Using the general FDM description, external loads can be applied to any direction through the vectors $\mathbf{p}_x$, $\mathbf{p}_y$, $\mathbf{p}_z$ [*n*x1], which are partitioned in $p_{x,i}$ and $p_{x,b}$ referring to the n_i free and n_b fixed, or constrained nodes in the network (analogously in y- and z- directions). Similarly, the connectivity matrix can be sliced in $\mathbf{C}_i$ [mxn_i] and $\mathbf{C}_b$ [mxn_b]. Using this definition and introducing the coordinate difference matrices **U**=diag(**Cx**), **V**=diag(**Cy**), **W**=diag(**Cz**) [*mxm*] the equilibrium equations in the free nodes can be rewritten for x-, y-, and z- direction as:

$$\begin{aligned} \mathbf{C}_i^T\mathbf{U}\mathbf{q} &= \mathbf{p}_{x,i}, \\ \mathbf{C}_i^T\mathbf{V}\mathbf{q} &= \mathbf{p}_{y,i}, \\ \mathbf{C}_i^T\mathbf{W}\mathbf{q} &= \mathbf{p}_{z,i}. \end{aligned} \qquad [9.1\text{-}9.3]$$

Finally, the coordinates of a network with given applied loads can be computed in terms of the positions of the fixed vertices $\mathbf{x}_b$, $\mathbf{y}_b$, $\mathbf{z}_b$ [n_bx1] and of the force densities in all edges **q**.

$$\begin{aligned} \mathbf{x}_i &= \left(\mathbf{C}_i^T\mathbf{Q}\mathbf{C}_i\right)^{-1}\left(\mathbf{p}_{x,i} - \mathbf{C}_b^T\mathbf{Q}\mathbf{C}_b\mathbf{x}_b\right), \\ \mathbf{y}_i &= \left(\mathbf{C}_i^T\mathbf{Q}\mathbf{C}_i\right)^{-1}\left(\mathbf{p}_{y,i} - \mathbf{C}_b^T\mathbf{Q}\mathbf{C}_b\mathbf{y}_b\right), \\ \mathbf{z}_i &= \left(\mathbf{C}_i^T\mathbf{Q}\mathbf{C}_i\right)^{-1}\left(\mathbf{p}_{z,i} - \mathbf{C}_b^T\mathbf{Q}\mathbf{C}_b\mathbf{z}_b\right), \end{aligned} \qquad [10.1\text{-}10.3]$$

where **Q**=diag(**q**).

Therefore, by using this general approach, all coordinates in a network can be controlled by modifying the force densities and the positions of the support vertices of the network.
However, controlling the specific position of the network using all **q** as parameters is a hard problem. The clarity imposed by the definition of the form diagram is lost in such general application; particular features such as creases, crack lines, and load-point applications can no longer be considered. Furthermore, dense diagrams might easily present more than 1000 edges, increasing the number of parameters of an optimisation. In the next section, a strategy for decoupling the horizontal and vertical equilibrium is presented and explored by defining the degrees of freedom of form diagrams fixed in plan.

3.2. *Degrees of freedom of a network with fixed projection*

To reduce the degrees of freedom of Equation 10 and to keep the form diagram fixed, a linear relationship is introduced to the force densities in the network. Assuming that **x**, **y** and **U**, **V** are known, the horizontal equilibrium equations 9.1 and 9.2 can be combined introducing the horizontal equilibrium matrix **E** [$2n_i$xm]. Similarly, the applied horizontal external forces are combined in the vector $\mathbf{p}_{\mathbf{h},\mathrm{i}}$ resulting in the following expression:

$$\mathbf{Eq} = \mathbf{p}_{\mathbf{h},\mathbf{i}}, \qquad \text{with: } \mathbf{E} = \begin{bmatrix} \mathbf{C}_\mathbf{i}^\mathbf{T}\mathbf{U} \\ \mathbf{C}_\mathbf{i}^\mathbf{T}\mathbf{V} \end{bmatrix}, \mathbf{p}_{\mathbf{h},\mathbf{i}} = \begin{bmatrix} \mathbf{p}_{\mathrm{x,i}} \\ \mathbf{p}_{\mathrm{y,i}} \end{bmatrix}. \qquad [11]$$

Finding a set of force densities satisfying Equation 11 means finding a distribution of horizontal forces in the fixed pattern. Furthermore, if we constrain all force densities to be negative, this solution will be compression only (Section 4).
The number of force densities that can be chosen freely in Equation 11 corresponds to the number of degrees of freedom (DOFs) of the fixed form diagram. This number is equal to the rank deficiency of the matrix **E** (Van Mele *et al.,* 2014). These are known as independent force densities $\mathbf{q}_{\mathrm{id}}$ and can be found through sequential singular-value decomposition as shown in Maia Avelino *et al*. (2021). Once the independent edges are determined, the set of dependent force densities can be computed with:

$$\mathbf{q}_\mathbf{d} = -\mathbf{E}_\mathbf{d}^\dagger\left(\mathbf{E}_\mathbf{i}\mathbf{q}_\mathbf{id} - \mathbf{p}_{\mathbf{h},\mathbf{i}}\right), \qquad [12]$$

where $\mathbf{E_d}$ and $\mathbf{E_i}$ are slices of $\mathbf{E}$ related to the dependent and independent edges, respectively, and $\mathbf{E}_d^\dagger$ corresponds to the generalised inverse or Moore-Penrose pseudoinverse of $\mathbf{E}_d$. Once $\mathbf{q}_d$ is computed from $\mathbf{q}_{id}$, the vector of force densities $\mathbf{q}$ in the system is retrieved through the linear transformation

$$\mathbf{q} = \mathbf{B}\mathbf{q_{id}} + \mathbf{d}, \qquad \text{with: } \mathbf{B} = \left[\frac{-\mathbf{E_d^\dagger E_i}}{\mathbf{I_k}}\right], \mathbf{d} = \left[\frac{\mathbf{E_d^\dagger p_{h,i}}}{\mathbf{0}}\right] \qquad [13]$$

where $\mathbf{I}_k$ is the identity matrix of size *k*. After such variable reduction, the vertical coordinates of the network are now function of $\mathbf{q}_{id}$ and $\mathbf{z}_b$:

$$\mathbf{z}_\mathrm{i}(\mathbf{q_{id}}, \mathbf{z_b}) = \left(\mathbf{C}_\mathrm{i}^\mathrm{T}\mathbf{Q}\mathbf{C}_\mathrm{i}\right)^{-1}\left(\mathbf{p}_{\mathrm{z,i}} - \mathbf{C}_\mathrm{b}^\mathrm{T}\mathbf{Q}\mathbf{C}_\mathrm{b}\mathbf{z}_\mathrm{b}\right) \qquad [14]$$

An initial approximation of the vertical loads ($\mathbf{p}_{z,i}$) can be computed at the beginning of the process. After the definition of the form diagram in plan, projected tributary areas per vertex can be directly obtained from it.
It is worth noting that this formulation can cope with horizontal external loads in the vector $\mathbf{p_{h,i}}$ as long as the form diagram is able to transfer the loads to the supports, which requires that the rank of $\mathbf{E}$ is not increased by concatenating $\mathbf{p_{h,i}}$ as a column (Bruggi, 2020).
For the cases where no horizontal loads are applied, a force diagram can always be retrieved from the in-equilibrium force densities (Van Mele *et al*. 2012). In fact, the degrees of freedom of the pattern corresponds to the possible modifications in the force diagram that will preserve the orientations of the edges of the form diagram (Block, 2019; Block, Lachauer, 2014). **Figure 6** shows such manipulations for the orthogonal form diagram used in the previous examples **[fig. 4]** with one set of independent edges highlighted in blue. For this diagram, the selection of the independent edges and their effects on the resulting internal force pattern is trivial: each continuous line must have one independent edge and thus an independent thrust value. **Figure 6** shows the modifications necessary to obtain Γ_1* **[fig. 4b]** and Γ_2* **[fig. 4c]**, where the (horizontal) force in the independent edge controlling the central continuous arch in the structure is increased by a factor of 4.0.
In general cases, especially for triangulated patterns, finding these independent edges and understanding their effect in the global horizontal equilibrium is not trivial. **Figure 7** shows a set of independent edges for the corner-supported form diagram Γ. Next to it, an individual increase in

the force magnitude for each independent edge is applied, and the force diagram Γ_i* and thrust network Gi obtained are depicted. The infinite combination of the individual effect of all independent edge represent the possible (horizontal) equilibrium states for the form diagram Γ, resulting in different thrust networks G.
The main advantage of the numerical formulation presented in this section is that the problem is now reduced in terms of variables - the independent edges ($\mathbf{q}_{id}$) and the heights of the fixed vertices ($\mathbf{z}_b$) - which is especially suitable for the optimisation algorithms presented in Section 4. In this formulation, the horizontal equilibrium in Equation 13 is decoupled from the vertical but they are computed simultaneously. Therefore, all possible horizontal equilibria are considered when searching for different networks within geometric bounds. As a consequence, the presented numerical approach enables to explore the full equilibrium space of the problem.

4. *Searching for admissible stress states*

This section presents the latest research on TNA in which the problem of searching for admissible stress states is encoded in a nonlinear constrained optimisation process. The overall nonlinear programming (NLP) problem and different objective functions are discussed in Section 4.1. In Section 4.2, a method to compute the level of stability of a masonry vault is presented.

4.1. *A nonlinear optimisation framework*

Finding networks with specific geometric configurations is a hard numerical problem because the heights of the network are nonlinear with respect to the independent force densities ($\mathbf{q}_{id}$) per Equation 14. The general problem that needs to be solved for the assessment of masonry structures, is presented below for a generic objective function f_{obj}:

$$\min_{\mathbf{q_{id}},\mathbf{z_b}} \quad f_{\text{obj}}(\mathbf{q_{id}}, \mathbf{z_b}), \qquad [15.1]$$

$$\text{s.t.} \quad \mathbf{q} = \mathbf{B}\mathbf{q_{id}} + \mathbf{d}, \qquad [15.2]$$

$$q_j \leq 0, \qquad \text{for } j = \{1, 2, \dots m\}, \qquad [15.3]$$

$$z_i^{LB} \leq z_i \leq z_i^{UB}, \qquad \text{for } i = \{1, 2, \dots n\}, \qquad [15.4]$$

in which the variables are the independent force densities $\mathbf{q}_{id}$ and the heights of the supports $\mathbf{z}_b$. The force densities in all edges are computed per Equation 15.2. Equation 15.3 imposes the compression-only constraints to all force densities of the network, which are constrained to be negative. Equation 15.4 imposes that the vertical heights of the network are contained within the envelope of the masonry described through the upper and lower bound heights, z_i^{UB} and z_i^{LB}, at each vertex *i*. Additional constraints and variables can be coupled to this modular optimisation framework.

The optimisation problem described in Equation 15 can be solved with general NLP solvers, such as interior point optimisation (IPOPT) (Wächter, Biegler, 2006), Method of moving asymptotes (MMA) (Svanberg, 1987), and Sequential Least Squares Programming (SLSQP) (Kraft, 1988). The derivatives and gradients have been computed analytically in (Van Mele *et al*., 2014; Bruggi, 2020; Maia Avelino *et al*., 2021).

Different works in the literature have assumed distincts objective functions to the general optimisation framework. In Block, Lachauer (2014) and Van Mele *et al*. (2014), the objective function selected is the “best fit”. This objective minimises the vertical least-square distances of the network’s vertices to given target heights. In these works, the explicit constraints on the bounds (Equation 15.4) are not considered.

In subsequent works (Bruggi, 2020; Maia Avelino *et al*., 2021), the bounds on intrados and extrados are explicitly considered and multiple objective functions are considered. In Maia Avelino *et al*. (2021), three objective functions are considered: minimising and maximising the horizontal reactions and offsetting the starting envelope to find the minimum thickness for the structure. These objective functions are used to assess the level of stability through the construction of the stability domain, as will be explained in Section 4.2.

It is worth noting that in Block and Lachauer (2014), Bruggi (2020) and Maia Avelino et al. (2021) no assumption is made on the distribution of the horizontal forces of the network. The horizontal equilibrium is computed automatically, so without the need to enforce bounds to the thrusts as in O’Dwyer (1999) or Marmo, Rossati (2017).

Most recently, in Maia Avelino *et al*. (2022), the complementary energy of the network is minimised for a given vector of virtual displacements applied to the supports, which is adequate to model the behaviour of masonry structures subjected to differential foundation settlements.

Further objective functions are also possible, e.g., maximising a horizontal multiplier of the applied loads can be incorporated into the present workflow. This is the static equivalent for computing a horizontal action such as an earthquake, which can be simplified to a horizontal load equal to a fraction of the structure' self-weight (Milani *et al.*, 2016; Nodargi, Bisegna, 2021).

Figure 8 illustrates different objective functions relevant for the assessment of vaulted masonry structures applied to a semi-circular arch, these that can be coupled to the optimisation problem in Equation 15. **Table 1** shows the objective function for each case and a description. This list is non-exhaustive and further work might expand it including new relevant objective function implementations.

Objective Function		f_{obj}	Description
a	Minimise horizontal thrust	$\sum_{n_b} \sqrt{R_{x,i}^2 + R_{y,i}^2}$	Minimises the horizontal component of the emerging reactions $(R_{x,i}, R_{y,i})$.
b	Maximise horizontal thrust	$\sum_{n_b} -\sqrt{R_{x,i}^2 + R_{y,i}^2}$	Maximises the horizontal component of the emerging reactions $(R_{x,i}, R_{y,i})$.
c	Minimise thickness	t	Minimises the structural thickness $(t > 0)$ computed orthogonally to the masonry's middle surface.
d	Best-fit	$\sum_{n} (z_i - z_i^t)^2$	Minimises the least-square vertical distance of the network to a target surface with heights z_i^t.
e	Minimise complementary energy	$\sum_{n_b} \boldsymbol{R}_i \cdot \boldsymbol{\phi}_i$	Minimises the complementary energy defined as sum of the dot product of the reactions $(\boldsymbol{R}_i \in \mathbb{R}^3)$ times the foundation displacements $(\boldsymbol{\phi}_i \in \mathbb{R}^3)$.
f	Maximise horizontal multiplier	λ	Maximises the horizontal load multiplier $(\lambda \geq 0)$ which applies a horizontal load proportional to the masonry's selfweight.

Table 1: Different objective functions that can be implemented in TNA.

4.2. *Computing the level of stability*

In a practical assessment scenario, assessing the level of stability of the structure is pressing. It implies answering how far the structure is from the collapse state, or how stable it is in its current configuration. However, finding one admissible stress state (as in the dome cap of **figure 2**) informs that the structure in its configuration is safe. Still, it does not provide information about the level of stability.

Furthermore, most of the optimisation objectives described in Table 1 alone can not give a quantity indicating the safety, i.e. the level of stability. A measure to compute the closeness to collapse on masonry structures was proposed in Heyman (1968, 1995) as the geometric safety factor (GSF). The GSF is defined as the ratio of the current thickness of the structure and its minimum thickness, i.e. the minimum thickness of the structure for which it is still stable. In Maia Avelino *et al*. (2021), the minimisation of the thickness is presented for analytic and non-analytic masonry geometries, resulting in the value of the GSF.
A more consistent measure of the level of stability is provided by defining the size of the space of admissible solutions. A reasonable measure of this domain is represented by its extreme (minimum and maximum) thrusts. For all but the limit state, the minimum and maximum thrust correspond to different stress states and have distinct (horizontal) thrust values. However, at the limit state, minimum and maximum thrust coincide. In Maia Avelino, *et al*. (2021), the stability domain is traced for reduced values of thickness, until the point of collapse. Understanding how the stability domain changes as a function of the thickness gives a direct measure of the robustness of the structure from its initial state, until the collapse state. This robustness can be associated with the structure's capacity to carry additional imposed loads or undergo external settlements.
By combining these two measures, a picture of the stability and robustness of the structure can be drawn, which is in accordance with the lower-bound, or safe theorem, in which the stability domain is approximated by the interior, i.e., by the safe side.

5. *Applications*

In this section, recent results obtained using Thrust Network Analysis are compiled to showcase the wide range of applications of the method to relevant masonry assessment problems. Section 5.1 shows the application of TNA for the computation of the stability domain and geometric safety factor of a hemispherical dome. Section 5.2 shows how a similar approach can be applied to general vaults, comparing different assumptions on force flows by evaluating different form diagrams. Finally, Section 5.3 shows an application of TNA in combination with an energy criteria, which might be used for the inverse analysis of structures subjected to foundation displacements.

5.1. *Assessment of a hemispherical dome*

The first example considers a hemispheric dome which will be described by the ratio between its thickness and its central radius t/R_c. The dome is assumed to have an initial thickness t_0/R_c=0.10 as is depicted in **figure 9a**. We assume the form diagram depicted in **figure 9b** to perform the TNA analysis. This diagram is composed of 20 hoops that are equally spaced in plan and 16 meridian segments that link the outer perimeter of the pattern to the centre. The final diagram is composed of 640 edges, for which a possible set of 33 independents are highlighted in blue in **figure 9b**. The shape of the thrust network is, therefore, a parameter of the 33 independent force densities and 16 vertical support heights, resulting in a total of 49 variables.
To study the stability of this dome, first, a minimum-thickness analysis is performed by computing the optimisation problem of Equation 15, having as objective function the direct minimisation of the thickness of the dome (Table 1c). The equilibrium result obtained is depicted in **figure 9c-9d** resulting in a t_{MIN}/R_c=0.041. In **figure 9c**, the edges of the thrust network that carry zero force are not shown, and the edges carrying (compressive) forces have their thickness scaled proportional to the forces carried. Points touching intrados and extrados are highlighted in blue and green, respectively.
A qualitative description of the minimum thickness solution shows that on the central part of the dome a bi-axial compressive cap is observed and a uniaxial stress state forms towards the supports, where the hoop forces vanish. Such internal force distribution is aligned with the "orange slice" mechanism proposed in Heyman (1988) for an outward (passive) radial displacement of the supports. The minimum thickness obtained in the TNA analysis is also in accordance with the theoretical minimum thickness of $(t_{MIN}/R_c)_{theory}$= 0.042, computed in Heyman (1988), presenting an error of less than 2%.
Comparing this minimum thickness to the initial dome thickness, the GSF of the dome can be calculated as 2.44, which allows one to conclude that a hemispherical dome with a given thickness over radius t_0/R_c = 0.10 is safe under its self-weight.
Further analysis of the stability of the dome can be provided by plotting the stability domain for the initial thickness t_0/R_c = 0.10. This is done by successively computing the minimum and maximum thrusts of the dome for decreasing offset thicknesses.

Figure 10 shows the result of such a process, where the values of minimum and maximum thrust, normalised by the dome's weight (T/W), are plotted as percentages in blue and red, respectively. The stability domain corresponds to the area between the two curves, highlighted in grey. In this area, one can find all admissible stress states of the problem for the given starting geometry and chosen form diagram. This stability domain is nonlinear and shrinks parabolically towards the limit state, giving an idea of the drop of stability for reduced values of thickness.
Points A, B, and C can be extracted from the stability domain; they represent the maximum and minimum thrusts in the original state and the point with minimal thickness (limit state). The maximum and minimum thrust-over-weight is equal to 62.6% and 19.9% while the value of thrust over weight for the limit state is 24.3%. Further discussion on this example and an extended mesh sensitivity study are available in Maia Avelino *et al*. (2021).

5.2. *Assessment of a gothic masonry vault*

The second example deals with a Gothic vault constructed parametrically from the cross-sectional parameters defined in **figure 11a**. The parameters assumed are the base length (l_0), central radius (R), springing angle (β), the thickness of the vault (t) and the effective span (s). The thickness of the vault is computed orthogonally to the middle surface, and the effective span is obtained from the ratio R/l_0 and the springing angle β.
The geometry of a vault obtained with $R/l_0 = 0.71$, $t/s = 0.05$, and $\beta = 20^\circ$ is depicted in **figure 11b** and will be used in this analysis.
Unlike the first dome example, suggesting a layout for the forces in the vault is not straightforward. For that reason, the three patterns of **figure 12** are considered. These patterns represent different force-flow assumptions within the structure. Pattern (a) is named orthogonal diagram and presents orthogonal segments that converge to main diagonals that transfer the forces to the supports of the vault. Pattern (b) is the fan-like diagram, which directly connects the supports of the structure and its central portion (spandrel). In pattern (c), diagonals are added to pattern (a), allowing additional possible paths to the supports, and the unsupported boundaries are curved inwards.
For all patterns, only the four corners are set as supports, and the independent edges are highlighted in blue [**fig. 12**].

The self-weight of the vault is lumped into the vertices of the pattern following a 3D tributary area calculation based on the projection of the patterns onto the middle surface of the vault multiplied by the thickness. For each form diagram, the minimum thickness and the stability domain of the Gothic vault are computed. The results are depicted in **figure 13**.
From the analysis of **figure 13**, we can see that the three different diagrams give a slightly different evaluation of the level of stability for the vault. The curved diagram [**fig. 12c**] yields the minimum thickness, i.e., the one resulting in the highest GSF of 2.5 and minimum thickness-over-span of 0.02. The three-dimensional minimum thickness solution obtained with this diagram is depicted in **figure 13**. The orthogonal and the fan-like diagrams result in reduced GSFs, which are (coincidentally) the same for both diagrams and equal to 2.1 (minimum thickness-over-span of 0.026). Therefore, by studying the stability of this specific Gothic vault conducting an analysis with both diagrams would be (equally) too conservative. Yet, the minimum overall thrust of the structure is obtained with the orthogonal pattern, which can be associated with the preferred flow of forces for an outward (opening) displacement of the vault.
In conclusion, when assessing structures with TNA, the analysis of multiple diagrams is crucial. Indeed, the structure can assume different force flows for different states (i.e., extreme thrusts and minimum thickness). This reflects how masonry structures are able to adapt to different support displacements, which induce different internal stress states, as also referred to by other authors as an elastic behaviour (Huerta, 2001). As the solutions are always safe, i.e., correspond to a lower-bound of the collapse state, the analysis can be performed with multiple diagrams, and the real domain of stability of the vault corresponds to the convex envelope of the multiple distinct domains obtained with different diagrams. A complete parametric study of this problem is available in Maia Avelino *et al*. (2021b).

5.3. *Compatible stress states for given foundation displacements*

The last example shows how TNA can be linked with an energy criteria allowing to compute compatible stress states for a given set of (virtual) foundation displacements. The example considered is the same hemispheric dome of Section 5.1 with thickness over central ratio t/R_c = 0.10. As shown in Section 5.1, the dome is safe under its self-weight.
However, in practical assessment scenarios, cracks are often observed in

supposedly safe structures, which require a proper analysis to avoid unnecessary closures and interventions (Ianuzzo *et al*., 2021). These cracks appear as a consequence of the unilateral (compression-only) nature of unreinforced masonry structures and arise in most cases due to foundation displacements. Measuring the exact magnitude or even identifying where the displacement is taking place is challenging. An energy criteria can be used to perform an inverse analysis that associates the observed crack pattern in the structure with an assumption of the actual displacements that occur at the base, which can be done by minimising the structure's complementary energy (Iannuzzo *et al*., 2021).

The complementary energy of the structure corresponds to the work of the reaction forces once a given displacement is imposed to the supports of the structure. By computing the minimum complementary energy of the structure subjected to a given set of displacements, a stress state can be found compatible with the given foundation displacement, which reveals the locations where cracks are most likely to form.

The optimisation framework in Equation 15 can be modified to perform such analysis by assuming as objective function the complementary energy for a given foundation displacement map (Table 1e). The foundation displacement map imposed in **figure 14a** corresponds to the spreading of the supports of a dome in two halves. The solution is depicted in **figures 14b-d**.

The solution obtained shows that for the spreading of the dome in two halves, the forces tend to accumulate in the Section B-B, in the middle portion orthogonal to the movement. In this Section, the force distribution assumes a maximum thrust behaviour, as can be seen in **figure 14c**. Along Section A-A, parallel to the movement, there's a clear depression in the centre of the dome [**fig. 14d**], suggesting the two halves tilting inwards and imposing higher pressure onto the arch along Section B-B. Similarly to the observation for the minimum-thickness case in Section 5.1, a uniaxial stress state forms towards the base of the dome, suggesting meridional cracks, while the bi-axial cap remains in place, now following the tilting provoked by the support movement [**fig. 14c**].

In a practical scenario, the crack pattern suggested by the analysis can be compared to the cracks observed. The TNA model can then be adjusted so additional analyses are performed to investigate the most likely reason for observed cracks to form.

6. *Conclusions*

In this overview paper, the formulation of Thrust Network Analysis (TNA) is revisited, focusing on its application to assessing unreinforced vaulted masonry structures. TNA offers a fast and flexible methodology to compute lower-bound admissible equilibrium solutions for a given masonry envelope. The internal forces are discretised and lumped in a network with axial-only forces along the edges and external loads and supports assigned to the vertices.

First, a graphic formulation of the method is presented for which the connections with ancient graphic statics methods is highlighted. Form and force diagrams are introduced, the former representing the projection onto the plane of the internal axial forces and the second representing the horizontal equilibrium of the former. This graphic-statics-based TNA framework is particularly relevant for interactive funicular form exploration in a forward design process.

Then, a numerical and more robust formulation is presented, which allows framing TNA in optimisation processes necessary for its application in a masonry assessment context.

The fixed form diagram offers the analyst additional control to model major geometrical features and/or structural discontinuities, such as creases, cracks or point loads of the masonry structure to assess.

Recent advances in the method are presented in which multiple objective functions are possible. These focused on framing the method as a tool for masonry assessment by providing a consistent measure of the level of stability represented by the computation of the GSF and the stability domain. The latter is traced by computing the extreme (minimum and maximum) thrust values of the structure ranging from its current state to the minimum-thickness state. Further developments coupled TNA with an energy criteria, which allow the study of compatible admissible networks for given foundation displacements.

Finally, the intense recent activity on lower-bound equilibrium methods, including continuous and discrete methods, such as TNA, suggests that more research will arise in searching for admissible stress states in masonry vaulted structures. Multiple challenges and open questions remain, such as how to overcome the specificity of the form diagram and provide engineers with adapted or automatically generated diagrams for the analysis.

Future work points to connecting the admissible solutions to energy-based methods and comparing them with general, rigid-block-based three-dimensional tools.
Moreover, open-source computational tools are being developed based on TNA and other lower-bound methods, which will increase the number of assessment tools available, benefiting other researchers and practising engineers in the field of structural preservation.

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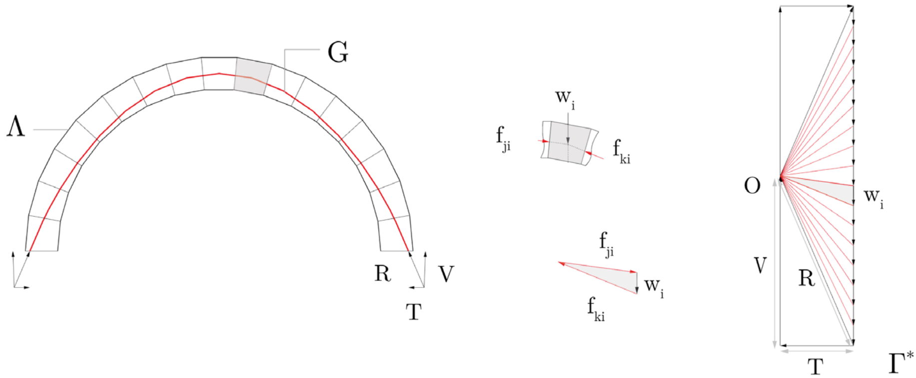

Fig. 1. Two-dimensional thrust line (G) within the semi-circular arch (Λ) highlighting the (equilibrium of) block *i* and reaction forces (R) with vertical (V) and horizontal (T) components. The global equilibrium is described by the force diagram (Γ*), which is composed by the multiple local equilibria in the nodes of the thrust line's vertices.

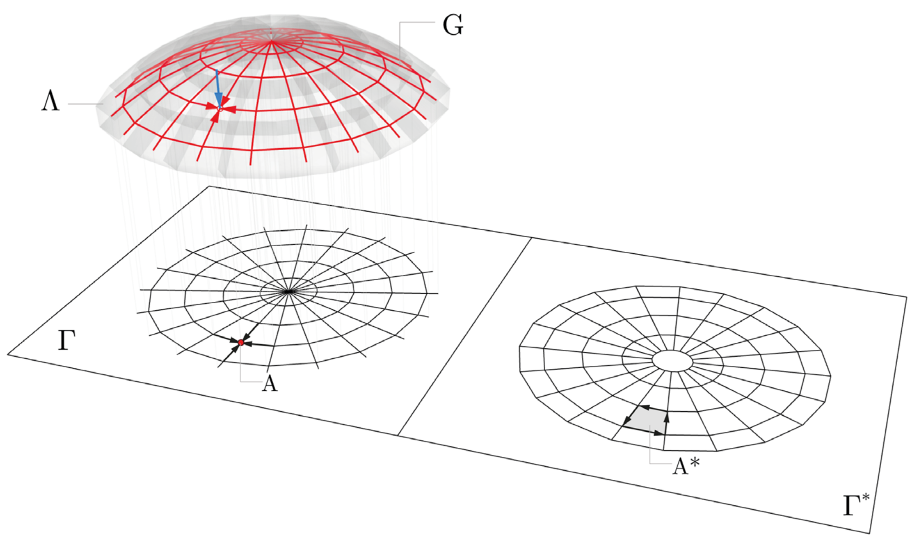

Fig. 2. A Thrust Network (G), its corresponding Form Diagram (Γ), defined as the horizontal projection of the thrusts, and the Force Diagram (Γ*), showing the equilibrium of the forces in Γ, which are the thrusts of G. The equilibrium of the (horizontal) forces applied to the highlighted node A in Γ is represented by the closed polygon A* in Γ*.

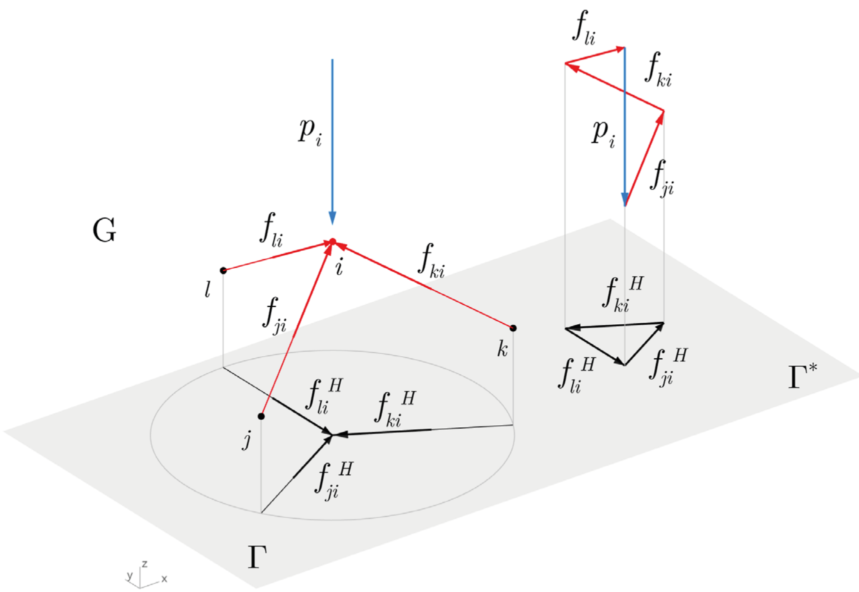

Fig. 3. Highlight of a node *i* in the network with an externally applied vertical load p_i. The compressive forces (f_{ji}, f_{ki}, f_{li}) in G can be decomposed in their horizontal (f_{ji}^H) and vertical (f_{ji}^V) components. The projection of the spatial equilibrium in the node results in the force diagram (Γ*) in which the vertically applied load p_i vanishes.

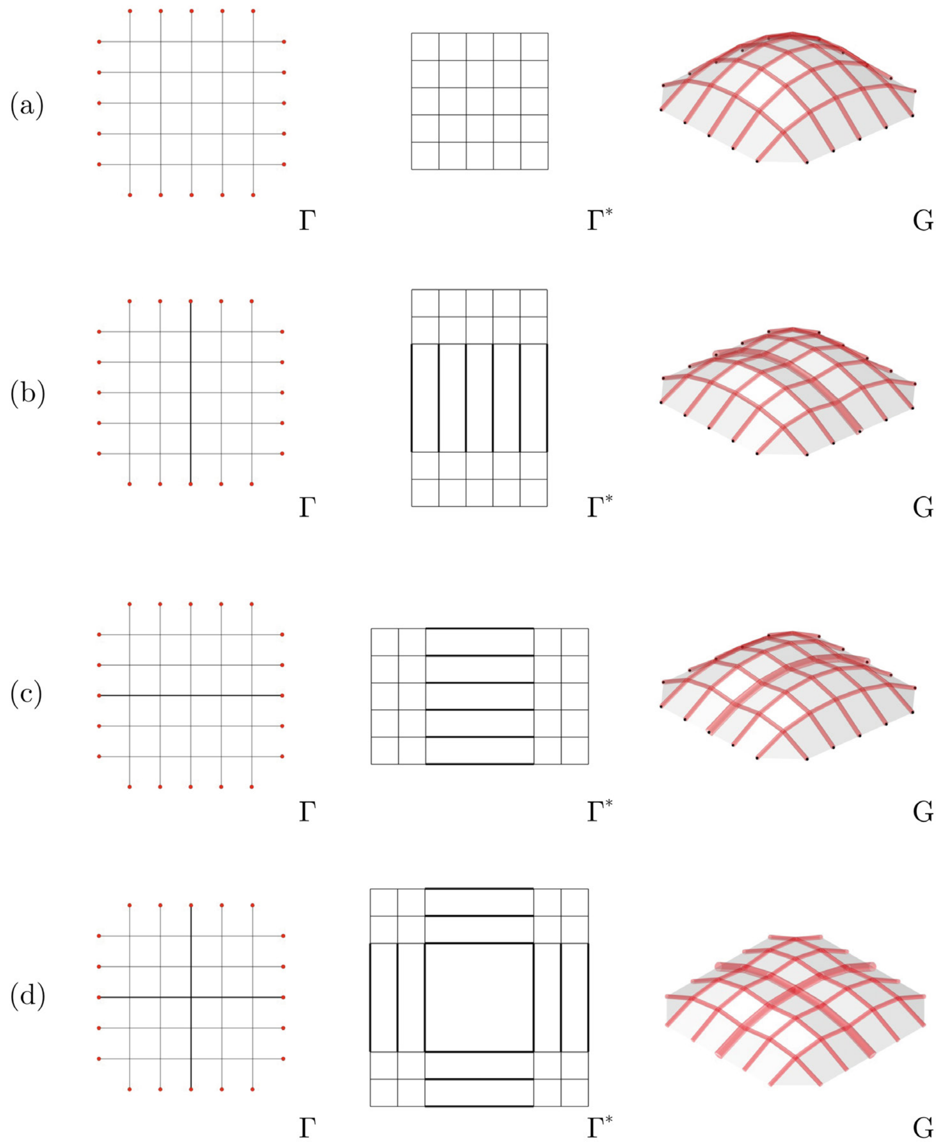

Fig. 4. Reciprocal form (Γ) and force (Γ*) diagrams, and the resulting thrust network (G) in an orthogonal grid supported along the boundaries subjected to four different horizontal force distributions. (a) Equally distributed forces, followed by (b) forces increased in the central vertical lines of the form, (c) central, horizontal lines, and (d) central, horizontal and vertical lines. Increased horizontal forces, i.e. longer edge lengths in the force diagram, result in shallow arches, i.e. creases in the thrust networks (G). In G, the thickness of the edges are proportional to the axial force carried.

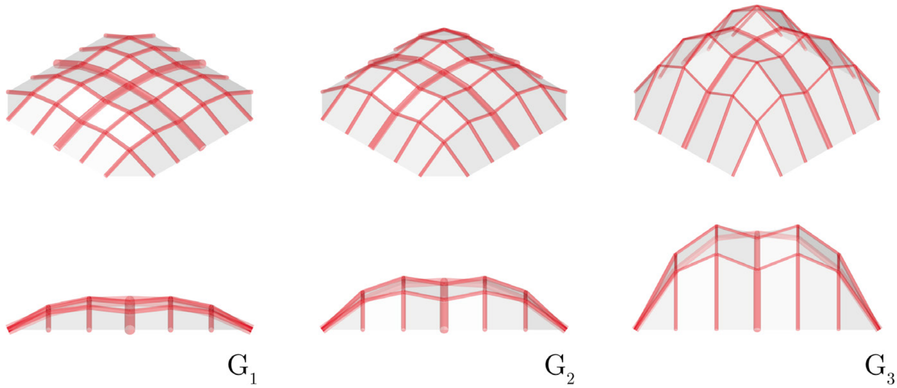

Fig. 5. Effect of the scaling factor $1/r$ in the height of the thrust network's vertices of Figure 4d. The scale factor for G is $1/r = 1$, and this factor is decreased for G_1 ($1/r_1 = 0.6$) and G2 ($1/r_2 = 0.3$) resulting in deeper networks keeping the same horizontal distribution of the forces (i.e., the force diagram of Figure 4d).

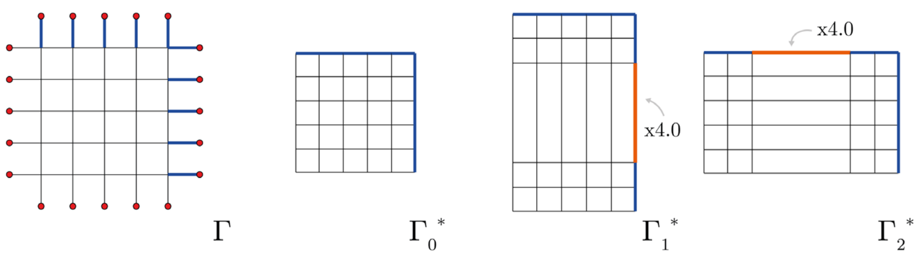

Fig. 6. Form diagram Γ with its independent edges highlighted in blue; its trivial reciprocal force diagram Γ_0* in which the forces are equally distributed and the dual independent edges are highlighted. Two modifications are performed to Γ_0*, multiplying the length of two independent edges by a factor of 4.0, which result in the force diagrams Γ_1* and Γ_2*.

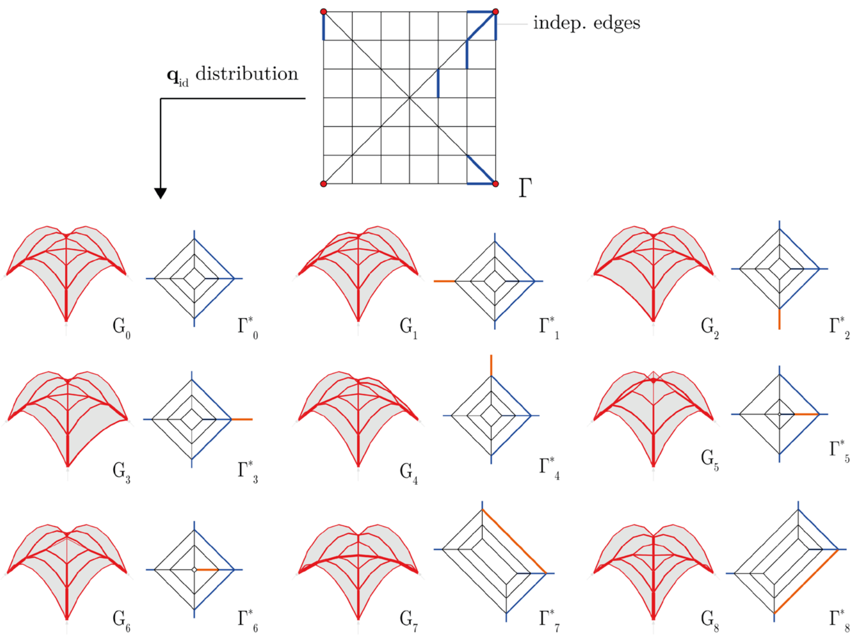

Fig. 7. Corner supported form diagram Γ with 8 independent edges (in blue). Initial force diagram Γ_0* and thrust network G_0, followed by an individual increase in the force of each independent edge, showing the effect in the force diagram Γ_i* and in the thrust network G_i.

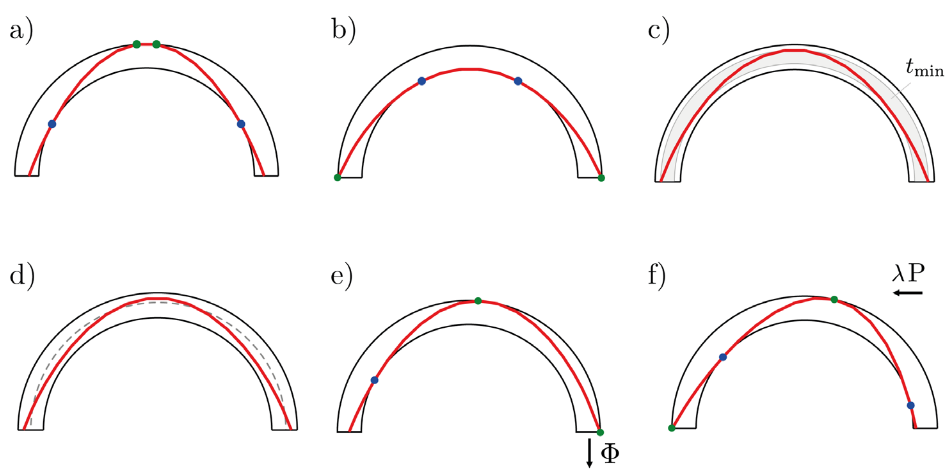

Fig. 8. Illustration on a semi-circular arch of the objective functions relevant to masonry assessment that can be explored with Thrust Network Analysis.

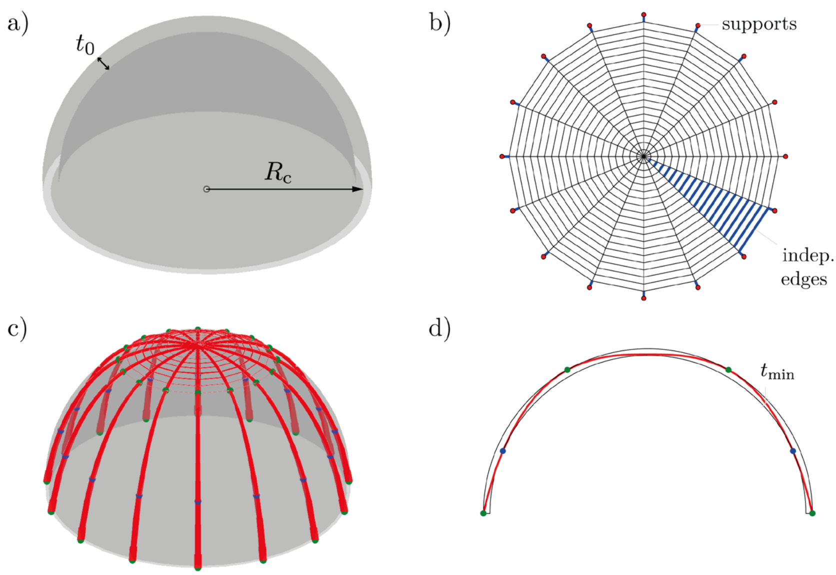

Fig. 9. (a) Perspective of the dome with initial thickness-over-radius t_0/R_c=0.10. (b) Form diagram used for the assessment of the hemispherical dome with support nodes in red and independent edges in blue. (c) Perspective of minimum thickness thrust network obtained with t_{MIN}/R=0.041. (d) Main cross section of the dome with highlight on points where the network touches intrados (blue) and extrados (green).

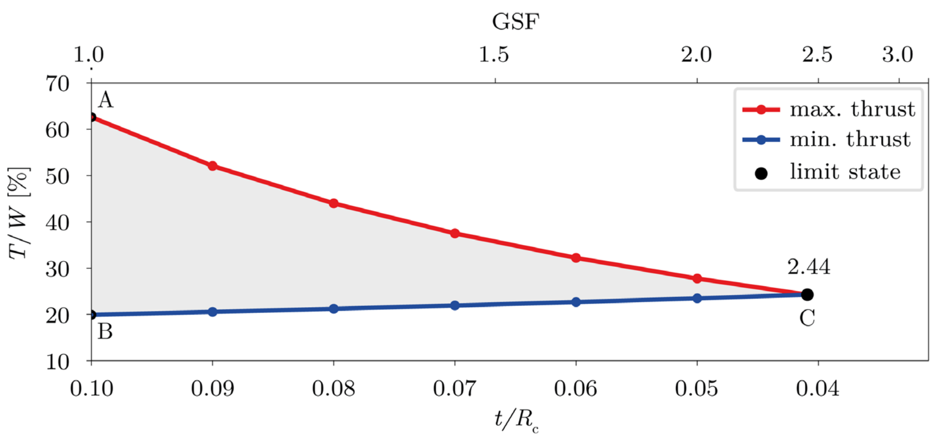

Fig. 10. Stability domain with values of normalised thrust-over-weight (T/W) for decreasing thicknesses of the dome, starting at t_0/R_c = 0.10. Point C highlights the limit state, corresponding to a GSF = 2.44. Points A and B highlight the maximum and minimum thrusts in the original state.

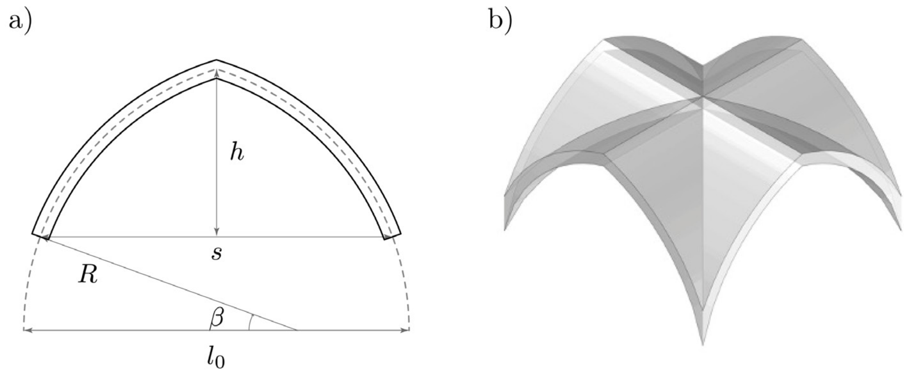

Fig. 11. (a) Parameters used to create the Gothic vaults in the study. (b) Perspective view of a Gothic vault constructed considering R/l_0 = 0.71, t/s = 0.05 and β = 20˚.

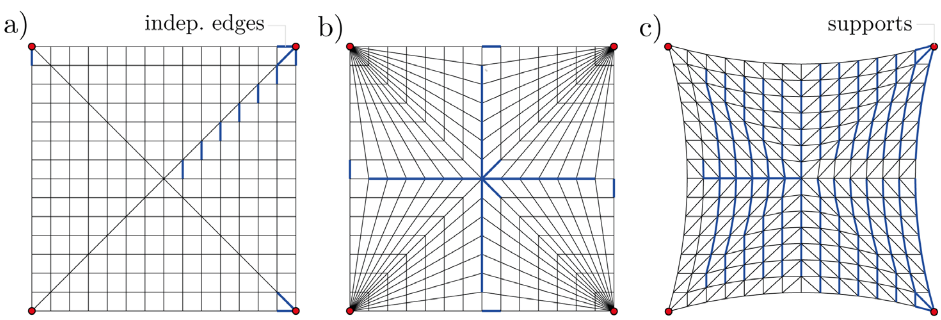

Fig. 12. Form diagrams considered for the analysis: (a) orthogonal, (b) fan-like, and (c) curved. Highlight on independent edges (blue) and supports (red).

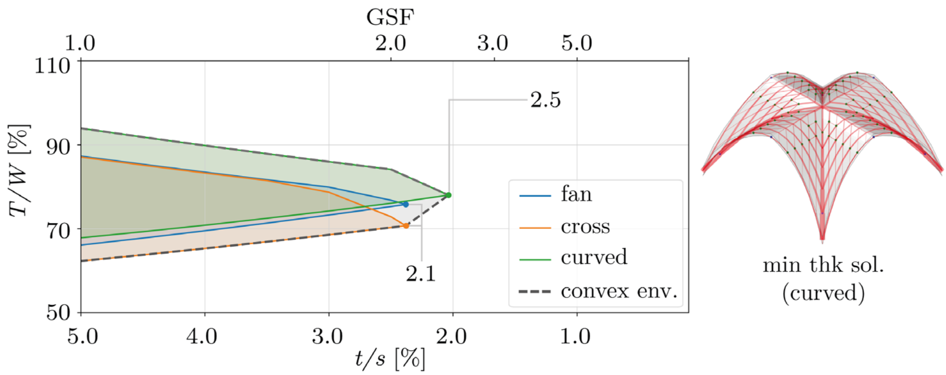

Fig. 13. Left: Stability domain for the different form diagrams in figure 12 and the convex envelope considered (grey). Right: Minimum thickness solution obtained for the curved diagram (t_{min}/s=0.021).

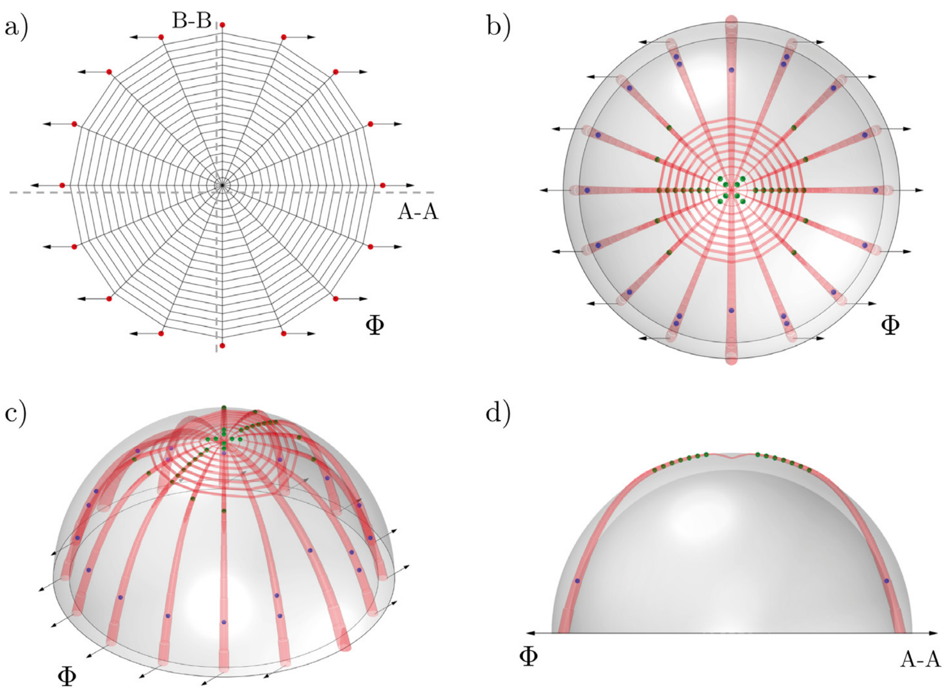

Fig. 14. (a) Assumed displacement of the supports (Φ). (b) Plan view of the found thrust network that minimises the complementary energy associated to Φ, highlighting the points where the thrust touches intrados and extrados. (c) Perspective view, and (d) sectional view of the same solution.

Stefano De Santis, Gianmarco de Felice

Load-carrying capacity and seismic behaviour of masonry arch bridges

1. *Introduction*

The infrastructural networks of several Countries all over the world present a rich heritage of masonry bridges, spacing from small single-span overpasses to large multi-span viaducts, which constitute a precious cultural heritage and evidence of the past. The activities of knowledge and valorization, conservation and assessment, inspection and diagnosis, repair, strengthening and retrofitting of existing bridges have recently experienced a wider and wider interest, highlighting the need of specialized analysis and intervention methodologies (ICOMOS, 2003).

On the one hand, masonry arch bridges generally show an extremely long life, thanks to the large self-weight, high strength and stiffness under exercise conditions. On the other hand, the design rules adopted in the past were based on empirical criteria or graphical methods. The expected traffic loads were lower than current ones and the seismic action was not considered in calculations. Finally, material deterioration processes, foundation settlements, damage, transformations or partial demolitions could have occurred with the passing of time. As a result, a deep awareness of the real safety level offered by existing masonry arch bridges is now lacking. Some issues on their structural behavior, such as their seismic performance, still deserve investigation. Tools for numerical simulation and structural analysis should be developed, that are suitable for engineering practice.

As for the load carrying capacity, several alternative approaches are available for the structural analysis of masonry bridges (Sarhosis, De Santis, de Felice, 2016). On the one hand, limit analysis-based methods can be used (see, among others: Harvey, 1988; Gilbert, Melbourne, 1994; Clemente, Occhiuzzi, Raithel, 1995; Boothby, 1997). They start from the assumptions proposed by Heyman (1966; 1982) that brickwork in compression is infinitely resistant or has finite strength with unlimited ductility, that it has no tensile resistance, and, finally, that no sliding between voussoirs occurs. Such assumptions simplify calculations, but may lead to an overestimate of the effective strength. On the other hand, applications of incremental finite element analysis have been proposed, making use of 1-D (Molins, Roca, 1998; Boothby, 2001; Brencich, De Francesco, 2004a; 2004b;

Brencich, De Francesco, Gambarotta, 2004; de Felice, 2009), 2-D elements (Cavicchi, Gambarotta, 2005; 2007; Gilbert, Nguyen, Smith, 2007) and 3-D elements (Fanning, Boothby, 2001; Fanning *et al.*, 2005; Harvey, Tomor, Smith, 2005; Domède, Sellier, 2010). As an alternative, expeditious empirical methodologies have been developed providing a quick estimate of the safety level of a bridge, such as the MEXE Method (UK Department of Transport, 1997; Wang, Melbourne, Tomor, 2010) and the SMART Method (Melbourne, Tomor, Wang*,* 2007).

In dynamics, static equivalent analyses based on the mechanism method are typically performed (Oppenheim, 1992; Clemente, 1998; De Luca, Giordano, Mele*,* 2004; De Lorenzis, DeJong, Ochsendorf, 2007), but the reliability of these approaches has still to be wholly verified. On the other hand, 3-D elasto-plastic finite elements (Pelà *et al.*, 2009) and 1-D non-linear macro-elements (Resemini, Lagomarsino, 2007) have also been used, which require much higher computational efforts and suffer from a high sensitivity to many input parameters. The seismic assessment of masonry arch bridges still deserves important research efforts.

Within this context, a research study was carried out at Roma Tre University on the mechanical behavior of historic brickwork and on the structural assessment of masonry arch bridges under traffic and earthquake loading. The study aimed at:

- giving a contribution to the knowledge of existing masonry arch bridges, starting from the mechanical properties of adopted materials;
- developing an approach for the structural analysis of masonry arch bridges using fibre beam elements, that is suitable for engineering practice;
- estimating the safety level of railway masonry arch bridges under traffic loading and discussing the influence of constitutive assumptions on load-carrying capacity;
- analyzing the seismic behavior of masonry arch bridges and discussing advantages and limits of the available methodologies, conceived for the design of buildings.

2. *Laboratory tests on historic brickwork under eccentric loading*

An experimental investigation was carried out in the laboratory to study the mechanical properties of the brickwork used in rail arch bridges under compression and bending, which is the stress condition experimented by the cross-section of vaults and piers (AlShebany, Sinha, 1999; Brencich, Colla, 2002; Brencich, Gambarotta, 2005; Brencich *et al.*, 2006; Roberts *et al.*, 2006; Brencich, de Felice, 2009). As a first step, some viaducts built between 1890 and 1894 along the rail line between Rome and Viterbo were surveyed. Based on information collected in the field, prismatic specimens were manufactured, which were provided with similar characteristics to the brickwork used in the bridges in terms of arrangement of the bricks, thickness of mortar joints, and bending axis. A total of 19 specimens of various sizes and geometries were built using historic bricks and poor lime mortar, and subjected to displacement controlled cyclic compression tests, with different values of loading eccentricity.

2.1. *Tests under compression*

Four masonry specimens were tested under cyclic centered axial load; the prisms measured 140×140×310mm^3 and consisted of five brick layers with 10÷12mm thick bed joints. Two specimens had a head mortar joint in the middle of the second and fourth layers, whilst the other two ones were made of five entire units, each made of half brick. A spherical articulation was used to ensure a uniform stress distribution between specimen and loading plates. The global displacements were acquired by means of four linear potentiometers, and the load was measured by the load cell integrated in the testing machine **[fig. 1a]**.
The response curves exhibited and initial elastic linear phase until the load reached 70÷80% of the maximum value. Then, vertical cracks appeared at about 80% of maximum load in a brick (often in the first or the last one, which were in contact with the loading plates) and then rapidly propagated into the other ones. The development of the first cracks was associated with a non-linear pre-peak phase. After the peak load, the cracks increased in size and involved the entire specimen, leading to a linear softening phase. The specimens after the end of the tests exhibited an evident transversal expansion of the central portion induced by some confining effect. The average compressive strength resulted f_M=7.7MPa for the specimens without head joints and f_M=4.2MPa for those provided with head joints. In this latter case, a preferential weakness plane developed, such that vertical cracks propagated starting from the vertical mortar joints and then involved the whole prism, leading to the lower failure load (-45%) with respect to that of the prisms without head joints **[fig. 1b]**.
The mean value of the initial tangent stiffness was 664MPa and 650MPa, whereas that measured on the reloading branches of the loading cycles

was 3090MPa and 2350MPa, respectively for the specimens without and with head joints. The stiffness in the reloading phases was higher than the slope of the first loading branch due to the compaction process developing in horizontal mortar joints, where the inelastic strain accumulated.

2.2. *Tests under compression and bending*

Three different specimen types were tested under cyclic eccentric axial load. The type was the same as for centred compression tests. It measured 140×140×310mm^3 and was made of five layers, each consisting of half-brick units. Two specimens were also provided with head joints in the second and in the fourth layers. The second type measured 280×140×310mm^3 and was made of five layers consisting of a brick unit alternating with two half-brick units; there was a head mortar joint in the second and in the fourth layers, parallel to the major side of the cross-section, which was also the direction of the axial load eccentricity. Finally, the third type measured 420×140×550 mm^3; it was made of nine layers of bricks as shown in **figure 1c**. It included a head joint parallel to the longer side of the cross-section in the second, forth, sixth and eighth layers, observable on the shorter sides of the prism. Head joints were staggered in the other direction. All the specimens were built so as to reproduce the typical arch brick arrangement and the way of applying the external load replicated the stress condition experimented by the cross-section of a bridge vault (Brencich, Morbiducci, 2007).

The eccentric axial load was applied by means of two steel bars with diameter $\varnothing$=20mm and two steel HEA140 I-beams, stiffened with vertical flanges, in contact with the bases of the masonry prism in order to apply compression with the desired eccentricity, avoiding local stress concentrations. The initial load eccentricity was known, since the stress resultant was forced to pass for the $\varnothing$20 steel bars. Displacement and strain data were measured using linear potentiometers. In particular, four potentiometers were fixed to the HEA steel I-beam corners, to measure global displacements and derive the macroscopic response; local deformations were measured by smaller transducers positioned vertically and horizontally on the central bricks.

Eccentric axial load tests lead to the determination of the response of a masonry element subjected to compression and bending, in terms of load-displacement and moment-curvature relationships. The global curvature χ (which was the average value along the element height) and the

global displacement δ (which was referred to the load application point) were computed from the displacement data acquired by the transducers applied to the corners of the steel I-beams. The applied load was measured by the load cell integrated in the testing machine. Finally, the bending moment was calculated considering second order effects deriving from the rotation of the steel plates.
The first crack appeared in the compressed edge at about 70÷80% of the peak load; then, cracks spread vertically crossing the mortar joints until they involved the whole specimen height, and small cracks also developed in the lateral faces. At the same time, one or more horizontal mortar joints opened in the opposite side. When deformation increased, a localization of cracks was detected, characterized by the crushing of one or two bricks on the compressed edge, and the opening of the corresponding horizontal bed joint on the opposite one **[fig. 1d]**.
The response curves of eccentrically loaded masonry specimens showed some similar properties if compared to the ones found in centered compression tests: an initial elastic branch followed by a non-linear pre-peak phase; then, after the displacement corresponding to the maximum force and the curvature corresponding to the maximum bending moment were exceeded, the behaviour was characterized by a linear softening phase. As in the case of centered compression, the cyclic tests showed the capacity of brickwork to sustain unloading-reloading cycles, also when performed in the softening branch. Cyclic loading induced a weak strength degradation, even if the skeleton curve could still be seen as the envelope of the cyclic one, and no heavy stiffness reduction was found. The compaction of mortar bed joints resulted in an increase of the overall stiffness; thus, the slope of the reloading branches was higher than the initial elastic one. More details on data analysis and test results can be found in (de Felice, De Santis, 2010).

3. *Numerical simulations with fibre beam models*

The modelling strategy proposed in this research for the analysis of multi-span masonry arch bridges makes use of a beam finite element with fibre cross-section, originally developed for the simulation of reinforced concrete members under seismic actions (Spacone, Filippou, Taucer, 1996). It is a non-linear element with distributed plasticity providing an accurate description of the structural response when distributed inelastic phenomena occur, thanks to an adequate representation of the interaction between axial force and bending moment on the cross-section under biaxial loading conditions. Moreover, it considers the highly non-linear hysteretic behaviour of the material, offering the possibility of performing structural analyses under complex loading histories. Finally, the simplicity inherent in a frame element assures relatively low computational and modelling efforts.

Analyses were carried out through the software OpenSees under displacement control. A corotational reference transformation rule and an energy increment-based convergence criterion were assumed. The fibre beam model was first calibrated based on centred compression tests. Then, its capacity to predict both failure conditions and whole response curve of masonry prisms under compression and bending was investigated. It was observed that the cross-section of the brickwork prisms tested under eccentric compression remained plane after deformation (de Felice, De Santis, 2010), which supported the choice of recurring to a beam element to model eccentrically loaded masonry elements.

Aiming at simulating the strength degradation observed in experimental tests on masonry specimens, a constitutive relationship was formulated and implemented in OpenSees. The skeleton curve was that of the Kent&Park model (Kent, Park, 1971), already available in the software, whereas the unloading-reloading phase was defined on the base of experimental data to capture damage accumulation and energy dissipation in the material due to cyclic loading. The compressive stress versus strain curve is plotted in **figure 2a** together with the experimental curve of brickwork prism under centred compression, showing that it yields a satisfactory representation of the effective response, also considering the strength degradation induced by cyclic loading.

The model is shown in **figure 2b**: the central beam represents the specimen and the rigid links on both ends represent the steel I-beam plates by means of which the load is applied, so as to consider the second order effects developing during the tests. Moreover, in order to reproduce the effective experimental conditions, two elasto-plastic no-tensile resisting elements were also included between the links and the beam, to simulate the detachment of the plate at the tensile edge of the specimen and the crushing of mortar at the compressed one. Numerical simulations predicted with good approximation the interaction between axial force and bending moment. The peak values of the experimental bending moment were predicted with a maximum mismatch of 8%. The whole cyclic force-displacement and bending moment-curvature behaviour of brickwork under eccentric loading was also simulated with good approximation, both in ascending and softening branches, with only a slight overestimate of the post-peak strength (it should be noted that, in the softening phase, the plane section assumption is no longer ensured). A comparison is shown in **figure 3c**, and the reader is refereed to (de Felice and De Santis, 2010), in which all the results are described and commented in depth.

4. *Load-carrying capacity of masonry arch bridges*

4.1. *Modelling masonry arch bridges by means of fibre beam elements*

The proposed modelling strategy for the structural analysis of multi-span masonry bridges makes use of the fibre beam elements described in the previous section, which were used to represent all the main elements of the bridge. More specifically, the vaults were described as segmental arches made of rectilinear beam elements; the piers simply as 1-D columns. The connection between two adjacent arches and the pier on which they are built on was modelled by rigid links to consider the effective relative positions of their central axes. Generally, no shear deformation was considered for the vault, whereas it was included in the pier properties to avoid an overestimate of the effective stiffness, especially for squat piers. The discretization of the cross-section into fibres, beyond the advantages in deriving the global section force-deformation law by means of the uniaxial material behaviour, already discussed in the previous chapter, also allowed the presence of different materials to be reproduced, as it may happen for the piers, which were often built with regular squared stones in the external leaf, whereas the internal core was made of rubble masonry with poor mechanical properties.
The backing was modelled by using horizontal truss elements having the same depth of the vault and connecting corresponding nodes of adjacent arches up to the appropriate height. It has to be pointed out that the interaction between adjoining vaults strongly depends on the height of the backing, which allows the activation of a multiple arch mechanism. Great attention has to be paid to original drawings and documents to check the effective dimensions of building details. Similarly, the abutments were described through an adequate number of horizontal truss elements, connecting the nodes of the lateral vault to as many perfectly fixed nodes. The number of beam elements and of fibres in the cross-section was determined through a preliminary mesh validation. A good compromise was

pursued between robustness and accuracy on the one hand, and computational sustainability and modelling simplicity on the other hand.

4.2. *Application to a case study: estimate of the load-carrying capacity of Ronciglione Viaduct*

A number of case studies were considered to validate the proposed modelling strategy and no exploit its features to discuss some important issues related to the load carrying capacity and the structural assessment of masonry arch bridges. One of the most relevant case studies was Ronciglione Viaduct, which belongs to the Roma-Viterbo rail line. This latter connects the stations of Roma Trastevere and Viterbo Porta Fiorentina and has a total length of 95.3km. It was designed in 1889 and built between 1890 and 1894. Because of the uneven ground, the line presents a large number of trenches, surveys, galleries, and viaducts with spans ranging from 10m to 25m. Ronciglione Viaduct is close to the final station of the branch going from Capranica to Ronciglione and is the most important work of the whole line; it is not in service any more but is still in good maintaining conditions.
The bridge has a rectilinear layout and is made of seven circular barrel vaults and six piers with a maximum height of about 35m, giving the construction a very slender aspect. The barrel vaults are built with clay bricks realized in a kiln in Nepi (near Rome) having dimensions $28\times14\times6cm^3$, and hydraulic mortar. The arches have 18m span, 9m rise, 1.07m thickness and 4.60m depth. All the piers have a vertical slope of 3.5% in longitudinal direction and varying from 5% and 6% in transversal one. The second and the fifth piers are provided with buttresses in the transversal direction and are 1.50m larger in the longitudinal one to carry asymmetric arch thrusts during the building phases; this device allowed the vaults to be made in three subsequent steps making use of the same wood centrings. The spandrel walls are 75cm thick and 11m high from the springers and are made of regular courses of tuff squared stones; finally, the backing height is about 4.70m.
The viaduct was modelled by using 100 fibre beams for each arch and 50 for each pier, in both cases the cross-section was discretized into 100×1 fibres. The backing and the abutments were described by using 17 truss elements. Incremental analyses were carried out under displacement control to evaluate the load-carrying capacity of the viaduct under concentrated travelling load.

Three constitutive laws were assumed for the material. One was provided with infinite compressive strength, another one with an elasto-plastic behaviour with crushing strength f_{cp}=7.1MPa and unlimited ductility, and, finally, the constitutive relationship described in the previous section.
The resulting curves are shown in **figure 3**, highlighting the great influence that the assumptions on material behaviour may have on the load-carrying capacity of a masonry arch bridge. More specifically, in this case, a significant decrease in capacity was found when assuming a constitutive relationship with finite compressive resistance and limited ductility, which is consistent with the outcomes of other studies in the literature (Brencich, Morbiducci, 2007; de Felice, 2009). In other terms, and assuming that the proposed constitutive relationship provides a relatively reliable estimate of the actual ultimate strength of the bridge, the load-carrying capacity was overestimated by about 70% when ductility was assumed as unlimited and by about 100% when the possibility that the material could fail by crushing was neglected.
Once the load-carrying capacity analyses were performed under concentrated travelling force, the rail traffic load defined by Eurocode 1 (CEN, 2005) was considered. It consists of four concentrated forces of 250kN each, at a distance of 1.60m one from each other. A distributed load can also be applied but it was not included here because this was found to be the most conservative choice. Indeed, the comparison between the results provided by the different loading conditions (single force, four forces) showed that a wider spreading resulted in a higher capacity. A measure of the bridge safety level was defined as the ratio between the ultimate load and the resultant of the design load (1000kN) and, in the present case, a safety factor equal to 2.7 was found.

4.3. *Safety assessment of different typologies of Italian large-span rail bridges*

The same modelling approach and structural analysis method applied to Ronciglione Viaduct was used to estimate the safety level of a representative sample of 50 Italian large-span railway masonry arch bridges, dating to the 19th and 20th centuries, distributed all over the Italian territory **[fig. 4a]**. The sample included single-span and multi-span bridges, shallow and deep arches, viaducts with slender and stocky piers, for which the geometric properties were taken from original design drawings and

calculation reports. Despite such variability, the main structural characteristics and construction methods recurred and complied with the design criteria suggested by historical treatises. The load-carrying capacity of the bridges and the corresponding safety factor with respect to rail traffic loads was assessed, to derive an overview on the existing stock of railway masonry arch bridges. Clearly, the fibre beam-based modelling approach introduced a number of approximations, so the outcomes should be considered as a preliminary assessment.
Considering different constitutive relationships, as already done for Ronciglione Viaduct, confirmed that an accurate description of the material properties may play an important role in the structural analysis of a masonry arch bridge. Among the considered sample, an average overestimate of the safety factor of about 30% was found when the post-peak deterioration, shown by the experimental response curves of compressed brickwork, was neglected. Limit analysis-based results may be unconservative for large-span brick masonry bridges, especially for those having low rise-to-span ratio.
Despite the simplified assumptions at the base of the design methods reported by historical treatises and the increase in traffic demands (today's design loads are 67% higher than in 1906 and 11% than in 1924), the railway masonry arch bridges included in the sample resulted safe under exercise loads, with an average safety factor of 5.8. The analyses provided a higher load carrying capacity for bridges made out of a single span **[fig. 4b]** or having stocky piers **[fig. 4c]**, suggesting that the assessment of an existing masonry arch bridge should not disregard the interaction between adjacent spans (Harvey, Smith, 1991; Melbourne, Gilbert, Wagstaff, 1997; Oliveira, Lourenço, Lemos, 2010).
Sensitivity analyses on mechanical and geometric parameters, such as material crushing strength and deterioration rate, ring thickness and backfill height, showed the strong influence of the ring thickness on the load carrying capacity. Moreover, bridges with shallow and slender arches resulted particularly sensitive to the local characteristics of masonry, due to the higher compressive stress in the ring cross section. Bridges with deep arches, instead, resulted sensitive to the backfill height, since it constrains the arch deflection and partially prevents the development of the most critical collapse mechanisms. The reader is referred to (De Santis, de Felice, 2014b) for more details on the analyses on the sample of masonry arch bridges.

5. *Seismic behavior of masonry arch bridges*

5.1. *Validation of the modelling approach in dynamics and linear analyses*

The modelling approach described in the previous sections and already adopted for the estimate of the load-carrying capacity, was also used for the seismic assessment of masonry arch bridges. As a first step, a single arch under pulse base acceleration was considered and its failure was evaluated by means of a finite element model with fibre beams; the results were compared to those provided by the mechanism method (Oppenheim, 1992; DeJong *et al.*, 2008) to verify the reliability of the fibre beam approach. **Figure 5** shows the comparison for arches with different slenderness ratios, and the reader is referred to (De Santis, de Felice, 2014b) for more results. In the plot, the pulse duration is on the x-axis and its acceleration amplitude is on the y-axis. The failure condition provided by the mechanism method is plotted with the solid lines, whereas the results of FE simulations are represented by the marks. A good agreement was found for all the considered configurations; even if for very short impulses, it may be difficult to find stable numerical solutions. Sensitivity analyses also revealed a dependence on the slenderness and on the size of the arch, as well as on the constitutive assumptions, especially for short impulses and high amplitude values.

Then, the case study of Ronciglione viaduct was considered. Its dynamic behaviour in the elastic range was first investigated and the results in terms of modal shapes and natural frequencies were compared to those provided by a linear 3-D finite element model, in which all structural elements were faithfully represented. In the 1-D model, vaults and piers were described by using fibre beam elements, whereas for backing and abutments truss elements with fibre cross-section were used, as it is illustrated in the previous section. When the dynamic behaviour is under investigation, the representation of spandrels and fill in terms of mass results to be essential, as well as the contribution of the former ones in terms of in-plane stiffness (Brencich, Sabia, 2007). Therefore, the mass of soil and spandrels was represented through point-masses connected by rigid links to the underlying vaults. The stiffening contribution of the spandrel walls (and, indirectly, the interaction between adjacent volumes of fill soil) was represented by horizontal and diagonal truss elements. The stiffness of the diagonal truss elements was reduced to 50% in the modal analysis, since they only work as struts and not as ties. The cross-section of vaults and

piers was discretized into 100×100 fibres, while the one of truss elements representing backings, spandrels and abutments was divided into 30×30 fibres. Finally, a shear flexibility was assumed for the cross-section of piers in both directions. The 3-D model was made of brick finite elements with 8 nodes. A Young's modulus E=750MPa and a self-weight γ=1650kg/m^3 were assigned to vault brickwork, according to experimental results, whereas E=750MPa, γ=1500 kg/m^3, and E=200MPa, γ=1500kg/m^3 were chosen for pier tuff masonry and fill soil, respectively.
The first ten modal shapes of Ronciglione Viaduct were obtained. The bridge conformation, characterized by high central piers, is such that the principal modal shape was in transversal direction and nearly symmetric, with all nodal displacements of the same sign and higher for the central spans. The corresponding period was T=1.48s. The following three modes in ascending order of frequency were again in the transversal plane and presented cross-shaped horizontal displacements. The first longitudinal mode was the 5th one. On the whole, modal shapes strongly relied on the height of the central piers, the stiffness of the second and fifth ones and from the stiffness of the arch, which appeared high enough to produce an interaction between the spans, but not so high to make the piers move always towards the same direction. A satisfactory agreement was found in terms of both periods and modal shapes, especially for main modes.

5.2. *Push-over analyses*

Push-over analyses were carried out under in-plane and out-of-plane loads with different distributions: one (#1) was proportional to nodal masses, another one (#2) was proportional to nodal masses times horizontal displacements of longitudinal and transversal principal modes for in-plane and out-of-plane forces, respectively. Finally, a third distribution (#3) was also adopted in longitudinal direction, with horizontal and vertical loads proportional to nodal masses and displacements of the fundamental in-plane mode. Forces were applied to nodes of arches and piers, as well as to those representing fill soil and spandrels, within an incremental quasi-static analysis which followed the application of the entire self-weight.
Material properties were described by means of the uniaxial constitutive model calibrated from laboratory tests on brickwork prisms under centred compression, which was assigned to the fibres of the cross-section of beam elements representing vaults and piers, and to the truss elements describing backings, spandrels and abutments.

Analyses were carried out under displacement control, adopting an energy increment-based convergence criterion and a linear geometric transformation rule. The resulting capacity curves are plotted in **figures 6b and 6c** for in-plane and out-of-plane directions respectively. In the plots, the control displacement (d_k) is on the horizontal axis and the normalized base shear on the vertical one. The control displacement was the horizontal component of displacement of the springer and of the crown of the central span for in-plane and out-of-plane analyses, respectively; it was measured, obviously, in the same direction in which loads were applied. Such a choice was based on the modal shapes found in the dynamic characterization, since these nodes exhibited the largest displacements in the eigenvectors. The normalized base shear was the resultant base shear divided by the bridge self-weight.
Lower resistance and stiffness were found towards transversal forces (out-of-plane analyses) and when a distribution proportional to the product of nodal masses and modal displacements (distribution #2) was used, since its resultant was applied in a higher position and produces a stronger bending moment at the base of the piers. Under in-plane loads, distribution #3 provided even lower stiffness and resistance. The mismatch in stiffness of the different capacity curves in both directions resulted from the different displacement fields produced by the imposed loads. If compared to the modal shapes, the closest deformed configuration was provided by load distribution #3 and #2 for longitudinal and transversal directions, respectively.

5.3. *Non-linear incremental dynamic analyses (IDA) under natural accelerograms*

The evaluation of the response of Ronciglione Viaduct under earthquake motion was made by means of incremental dynamic analysis (IDA, Vamvatsikos, Cornell, 2002). The method consists in performing repeated non-linear dynamic analyses under accelerograms with increasing intensity level (IL), which is a monotonic scalable ground motion severity measure. In this case, the peak ground acceleration (PGA) was assumed as the intensity level measure and ranges from 0.2 to 2.0 (full application corresponds to IL=1.0).
A set of 14 natural accelerograms was selected from the European Strong Motion Database (ESD) and used as assumed as input signals (seven events, two components each). They were chosen and scaled to be compliant

with the ultimate limit state acceleration response spectra, i.e. to achieve a maximum spread from target spectra equal to ±10% in the 0.15÷2.50sec range (Iervolino *et al.*, 2008). The 14 signals were chosen among events having moment magnitude at least equal to 6 and distance between the source and the recording station at least equal to 20km, to avoid weak events and near field registrations. Moreover, only signals recorded on soils having characteristics similar to the foundation soil of Ronciglione Viaduct, which is classified as B according to Eurocode 8 (CEN, 2005) were chosen. A Rayleigh viscous damping was used for dynamic analyses.
As regards the output variables, the maximum control displacement (d_k) and the corresponding (same instant) resultant base shear (V_b) were recorded for each simulation so as to obtain a set of couples (d_k,V_b), useful for the comparison with push-over analyses. The control nodes were the springer and the crown of central span for in-plane and out-of-plane analyses, respectively, as it was done for non-linear static analyses. For each intensity level (IL), the average of displacement and base shear values resulting from the application of all the accelerograms are determined, and IDA curve was built. It is noteworthy that push-over curves were nearly symmetric and no heavy directional effect was found in time-step analyses; thus, the comparison was made in the only positive semi-plane d_k-V_b. The points representing the single dynamic simulations were widely scattered (especially in transversal direction), but average IDA curves retraced with good approximation the capacity curves for both longitudinal **[fig. 6b]** and transversal **[fig. 6c]** analyses, if an adequate horizontal load distribution was applied (#3 and #2 for in-plane and out-of-plane analyses, respectively). In **figures 6b and c,** the points on IDA curves relative to IL=1.0 represent the expected average response of the bridge under the various seismic scenarios, which was named as Performance Point.

6. *Conclusions*

The research study described in this work contributed to the knowledge of the mechanical behaviour of masonry arch bridges, starting from the material properties, which were derived by means of an experimental investigation, to the structural analysis, for which the use of a modelling approach based on fibre beam elements was adopted for load-carrying capacity evaluation and seismic assessment. The proposed approach is highly effective for preliminary assessment, but cannot replace the activities of visual inspection and survey of existing damage and the detailed structural analysis that are required in the management of existing bridges. The main conclusions are herewith summarized:

- The mechanical properties of historic brickwork used in the construction of arch bridges were investigated by means of cyclic tests under centred and eccentric compression. The compressive strength resulted to be between 4.5 and 7MPa, depending on the brick arrangement, and the average stiffness was in the order of 650MPa. A constitutive relationship to represent the cyclic response of brickwork was defined and its parameters calibrated based on experimental tests.
- A beam model with fibre cross-section was used to simulate laboratory experiments on brickwork prisms under cyclic eccentric compression. The comparisons between test outcomes and numerical simulations demonstrated the reliability of the modelling approach in representing masonry elements under axial force and bending moment. The approach appeared suitable for modelling masonry bridges, ensuring, at the same time, low computational costs and simplicity in the determination of the parameters.
- A sample of 50 historic rail masonry arch bridges with different geometric properties was selected. Structural analyses showed that an accurate description of the material properties (crushing strength, post-peak behaviour) may play an important effect on the estimated load-carrying capacity. Among the bridges under study, a significant overestimate of the load-carrying capacity, in the order of 35%, was found when assuming an unlimited ductility of the material, as in yield design methods. Results also indicated a higher load-carrying capacity for bridges with shallow arches, thick vaults or low thickness-to-span ratio, and for bridges made of a single span or having squat piers. Analyses for the assessment of the load-carrying capacity of multi-span masonry bridges under design loads (exercise conditions) also suggested to consider only the four concentrated forces provided by the code for rail traffic, neglecting the distributed load, which may produce a load-bearing increase.
- The dynamic response of a single arch was investigated leading to an estimate of the failure condition under impulse base motion in agreement with that provided by the mechanism method. Sensitivity analyses revealed a dependence on the slenderness and on the size of the arch, as well as on the constitutive assumptions, especially for short impulses and high amplitude values.

- The use of current methodologies for the seismic assessment of masonry bridges were examined with reference to an existing rail multi-span viaduct. The comparison with a 3-D finite element model revealed a good agreement in terms of natural frequencies and modal shapes. Cyclic push-over analyses under different load distributions, in both longitudinal and transversal directions, showed lower resistance and stiffness under out-of-plane forces. The comparison of push-over with incremental non-linear dynamic analyses (IDA), carried out under sets of 14 natural records, provided a good agreement when loads proportional to inertial forces were used.

This research study faced some of the most important issues in the structural analysis of masonry bridges, bringing to light the main problematic points related to these features and motivating numerous areas of continued research. One of the main improvements of the proposed modelling approach is represented by a more accurate description of spandrel walls and fill soil, whose contribution to the structural response is definitely non-negligible. One way could be to use 2-D multi-layered elements to describe, separately, walls and soil, treated as equivalent homogenized continua. The achievement of an accurate representation of the seismic response of multi-span masonry bridges deserves further investigation, which cannot prescind from experimental activities to determine the effective contribution of some structural elements (like the spandrel walls) to the overall response, the damping parameters to assume in non-linear dynamic analyses and the threshold values for the structure state variables that synthetically describe the response performance. Finally, an effort should be made in the field of the seismic assessment of masonry bridges on performance basis, which is definitely one of the most interesting and promising research branches for the next future.

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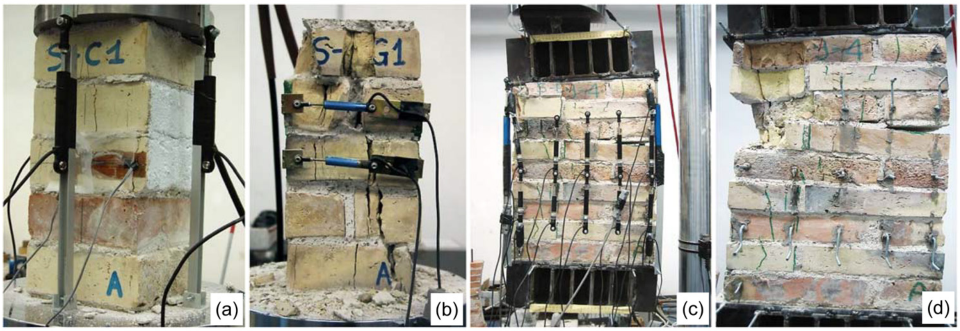

Fig. 1. Crack pattern of the brickwork specimens without head joints (a) and with head joints (b) tested under centred compression and crack pattern at peak load (c) and at failure (end of the test, d) of the brickwork specimens tested under compression and bending.

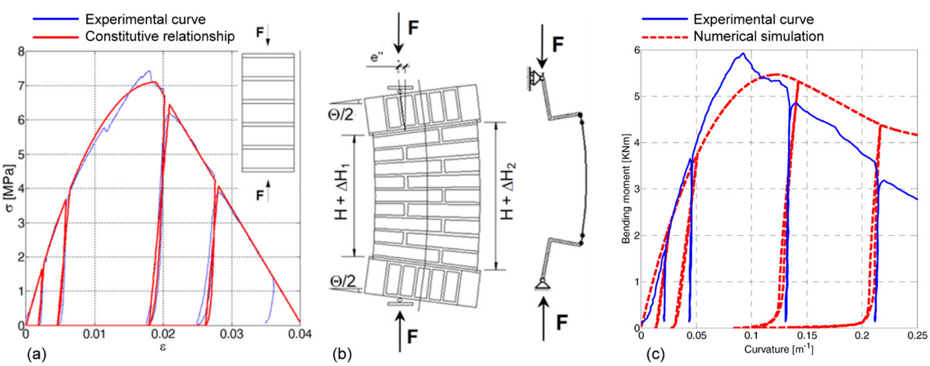

Fig. 2. Experimental response curve and constituive relatioship of brickwork under cyclic centred compression (a), fibre beam model (b) and experimental response curve and numerical simulation of brickeork under cyclic eccenetric compression.

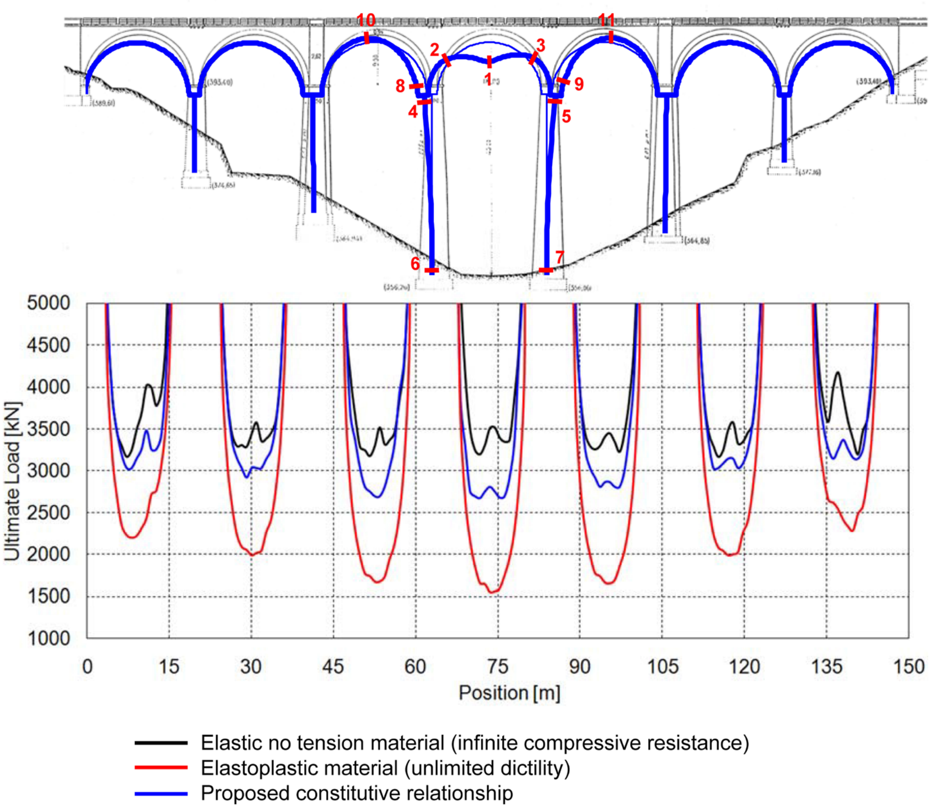

Fig. 3. Load-carrying capacity of Ronciglione Viaduct under concentrated travelling load for different material constitutive relationships.

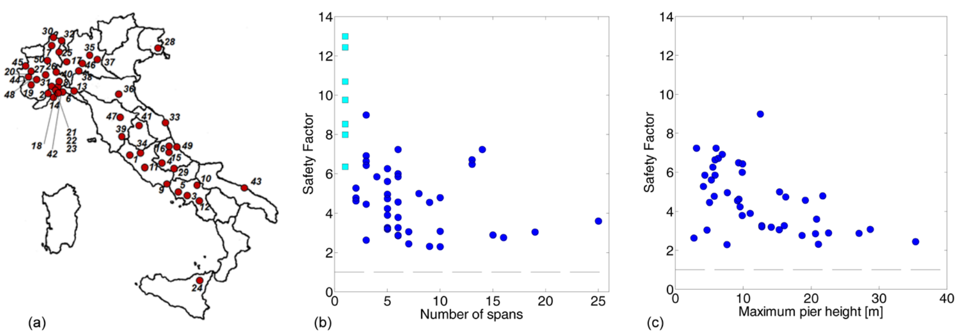

Fig. 4. Map bridges on the Italian territory selected for the sample (a) and dependence of the safety factor on number of spans (b) and on maximum pier height (c).

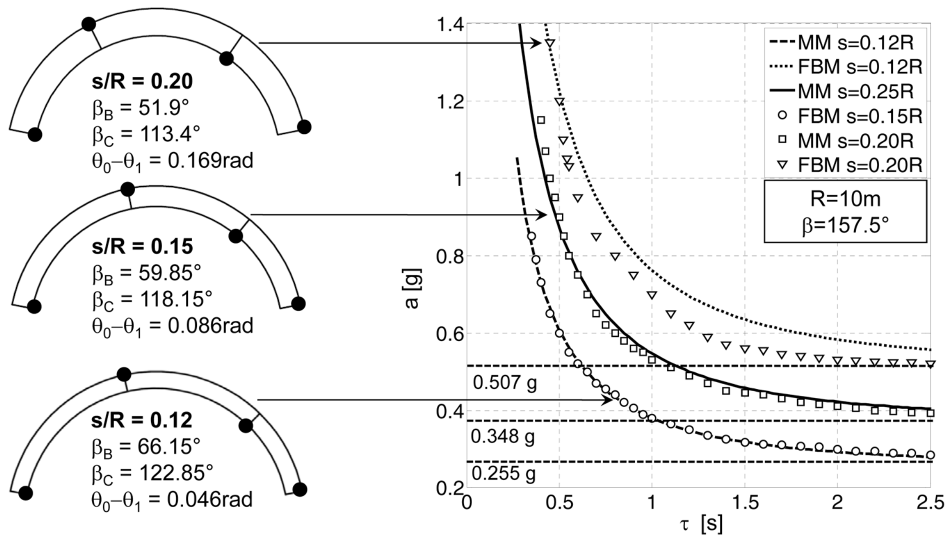

Fig. 5. Collapse domains of arches with radius R=10m, angle of embrace β=157.5° and thickness ranging from s=0.12 to s=0.20R under pulse base acceleration: comparison between mechanism method (MM) and fibre beam approach (FB).

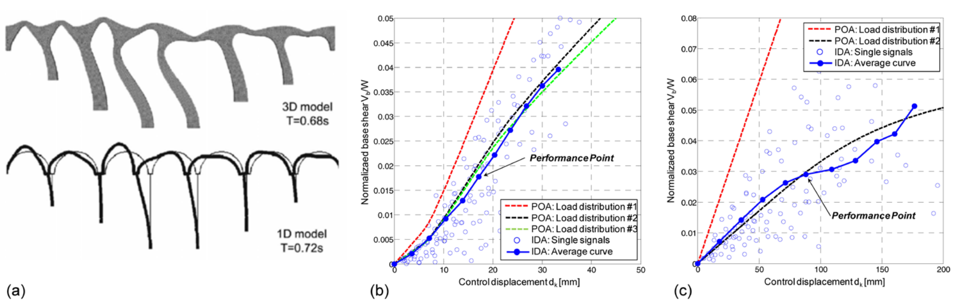

Fig. 6. First in-plane mode of Ronciglione Viaduct provided by a 3-D model and by the fibre beam model (a) and response curves of push over analyses (POA) and incremental dynamic analyses (IDA) for the seismic assessment in in-plane (b) and out-of-plane (c) directions

Christian Kayser

Freiburg and the consequences. Construction and reception gothic tracery spires

The "most beautiful tower on earth": Swiss art historian Jacob Burckhardt's much-cited epitheton of of the western tower of Freiburg minster acknowledges its special position in the history of medieval architecture. The approximately 116-metre-high building **[fig. 1]**, completed at the beginning of the 14th century, is one of the most important structures of gothic architecture as well as one of the tallest buildings of its time. Above all, its master builders succeeded in implementing a unique, bold design concept: the tower's stone spire, which is over 45 m high, consists entirely of tracery elements. Its interior forms, together with the open, high octagonal hall underneath, a spectacular architectural setting – a space, almost 60 m high, with a hull of lace-like stone ornamentation. The creation of the Freiburg openwork spire is one of the pivotal points in art history where, like Athene emerging fully armour-clad from the head of Zeus, a certain building typology is artistically as well as structurally fully realised in the first instance. The construction volume of the spire was not surpassed in subsequent medieval buildings, and to this day it is also the openwork spire with the best-preserved historic fabric. About 40 medieval spires follow its model. For about 250 years as well for numerous neogothic copies of the 19th century, the prodigious Freiburg tower was the building typology's main reference.
The western tower of Freiburg cathedral is also origin and reference point of my actual research project, a project that has occupied me for over a decade. It forms the follow-up to my doctoral thesis on the construction of gothic window tracery (Kayser, 2012), which was honoured with the Premio Edoardo Benvenuto XI. Whereas at the time I investigated the building construction and structure of planar gothic tracery constructions, i.e. windows, the focus of my actual research has shifted – or advanced – to spatially elaborate pyramids made of stone tracery: from plane to spatial object, from 2D to 3D.
At the beginning of the project stood an emergency, which turned out to be a stroke of luck (at least for me). During a routine inspection at the Freiburg tower irritating cracks were detected at the struts of the spire –

over 100 metre above the busy market place an alarming situation! As a result, the Freiburg building lodge ("Münsterbauhütte") – and the State Office for the Preservation of Monuments (Landesamt für Denkmalpflege Baden Württemberg) put together a team of experts to investigate the causes of the damage: One of the most elaborate and intensive investigation projects in recent monument conservation began. This flagship project could be completed in 2019 with the successful structural safeguarding of the spire. For me as a member of the team, the project was also the initial point for my research, which will be completed in 2022-2023 – a work on the openwork spires of the late Middle Ages, rooted in my research on window tracery of the gothic period.

The Model: the spire of Freiburg Minster

Key data of the building history

The data base for the reconstruction of the building sequence is – typical for that period – sparse. A possible testimony from the time of the tower's commencement are the inscriptions on its base. They represent local weights and measures, and give the dates 1270, 1317 and 1320[1]. These (somewhat uncertain) data are supplemented by the dendrochronological dating of the bell frame (1290-1291) and the main nave's roof (1301-1304), which leans against the tower's eastern flank.
In a rough sketch, the building sequence can be reconstructed with this: After construction began before 1270, the tower was built up to above the later organ gallery within approximately 15 years. Here, about four stone layers below the gothic cornice, we find indications on the outer walls that the construction was interrupted or perhaps even rescheduled. The joint can be discerned on the three "free" sides of the tower in the alternation of the ashlars' colours and formats and on the east side as a caesura between elaborately worked ashlar and rough masonry. It is also manifest in the connection of the western flying buttresses to the main nave's eaves. On the buttresses' extrados, originally gutters had been executed. Subsequently, these hewn channels were built over with the masonry of the eastern tower corners.

[1] Discussion of inscriptions' datations: Böker *et al.*, 2013, p. 76.

In 1291, the bell frame was erected. Despite warfare between the Counts of Freiburg and the city, decade the tower masonry advanced up above the height of the main nave's vault during the following. This is also confirmed by the mention of "nüwen turne[s], da die gloggen inne hangent" [i.e. the new tow in which the bells are hanging] in 1301 (Albert, 1907, pp. 29-77).

It must remain open how fast the construction work on the upper half of the tower progressed. However, a relatively rapid completion might be indicated by the fact that the construction volume of these parts is considerably lower than that of the mighty substructure. Accelerated work was necessary: The scaffoldings and falsework required for the construction of the spire were heavily exposed to the weather (Fritz, 1926, FN 88 and p. 229f). Damage to the scaffolding could only be repaired and supplemented with difficulty; moreover, with time passing, the supported stone structure would have become increasingly unstable. The haste with which the construction was completed is illustrated by details of the execution: In the spire's upper half, individual pieces of freestone were not even finished. A comparison with later openwork spires also testifies to the conjecture that the octagon and the spire could have been completed in relatively short span. The spire of St. Stephen's Cathedral in Vienna, which comes close to the volume of Freiburg, was completed within five years, from 1428 to 1433 /Uhlirz, 1901-1902; Zykan, 1970). At the tower of Brussels Town Hall, Jan van Ruysbroek created a three-storey octagon with a spire in six years (1449-1455) (Des Marez, 1923, p. 20), Hans of Cologne (Juan de Colonia) required for the northern spire of Burgos cathedral just about two years (1456-1458) (Menéndez González, 2016, p. 127).

In summa, the tower of Freiburg Minster was probably completed around 1320, thus corresponding with the dates inscribed at its base – the conclusion of the work constitutes the starting point of its restoration history (King, 2014).

Overview of the inventory

Layout of the tower

The 116-metre-high tower is divided externally into three main sections:

- Substructure with buttresses over rectangular ground plan, approx. 37,5 m high up to the star gallery
- Middle section, with transition from rectangular plan to octagon,

culminating in the open Octagonal hall approx. 32.5 m high from the star gallery to the spire gallery

- Tower spire over octagonal ground plan, approx. 45.5 m high

Remarkably, this distinct division into three parts does not correspond to the inner partition of the tower:

- Portal vestibule in the substructure, height 14 m up to the level of St. Michael's gallery / the intrados of the main nave.
- St Michael's gallery (organ gallery) in the substructure, height 15 m to the apex of the vaulting
- Bell house, with a clear height of approx. 25 m
- Octagonal hall, height approx. 14 m
- Tower spire over octagonal ground plan, height approx. 45.5 m

The discrepancy between the inner and outer structure is expertly processed at the level of the bell house. Inside, the bell house is built over a square ground plan, while the exterior appears, through skilful modification of the wall surfaces, to be part of the octagonal middle section of the tower.

<u>Fabric and structure of the spire</u> (Kayser, 2014)

The octagonal hall forms the basis of the towering openwork spire. In the interior, there is no caesura between the two components; thus, an enormous, open space up from the hall's floor to the top of the spire is created **[fig. 2]**. At the corners, the hall is accompanied by small elaborate finials. In the southeast, a polygonal stair turret is attached to the complex of bell house and octagonal hall. It runs over a height of 43 m and leads up the final gallery.

The octagonal hall is fitted with open tracery windows on each side. Above the capitals of the tracery columns, a wrought-iron ring anchor encircles the hall. The gallery (tower walkway) is laid out on its wall crown. Below the base of the spire, another wrought iron ring anchor is let into the crown masonry of the octagonal section. The actual spire is designed as an eight-sided pyramid. Six of the eight faces have approximately the same basic width (4.9-5.0m) and an identical design. The north and south sides are built over a slightly shortened base width (approx. 4.3m); the tracery design differs in details.

Phenomenologically, the spire is an openwork tracery pyramid with eight main struts at the corners and seven registers / storeys of intervening tracery panels. With the exception of the somewhat different first storey (see below), the principal structure is identical on all levels. It consists of only four different structural elements **[fig. 3]**:

- Tracery fields, composed of individual freestone elements
- Corner struts, made of similar, wedge-shaped corner stones
- Crockets on the corner struts
- Wrought iron ring anchors

The tracery fields butt up against the corner struts. Ring anchor systems run in the upper chords of each tracery storey. The wrought iron anchors are inserted into rectangular recesses / "grooves" in the upper chords of the tracery fields. In the corner stones, the end pieces of the iron bars are forged into hooks. These hooks are each wedged in wrought iron rings. The connection is sealed with cast lead. At the top of the tower, the struts merge to form a closed stone pyramid crowned by the ca. 3.5 m high finial. A historic plan drawing (Trost, 1843) provides information about its construction: The stone elements are, like a ´Shashlik´, threaded out on a wrought-iron "skewer".
The tower was built following an expertly devised and also economically sensible "modular system". The identical workpieces could be easily prepared in in the building lodge in winter and assembled on site in summer. Only the construction of the first, lowest storey of the spire with its sequence of magnificent rosettes deviates from the system described above. It was the first part to be implemented. Instead of a separation of tracery fields and struts, a very complicated stereotomy was realised: The freestones are set like a "3D puzzle" in an interlocking pattern, extending equally over struts and tracery. This results in intricate connections between the individual elements, which had to be planned and moved with the utmost precision at this level – a remarkable, astonishing achievement of the master builders and stonemasons! The modification of the construction method between the first and the following storeys might probably be rooted in the wish to economise and accelerate the construction process.

The Freiburg spire's reception

When Freiburg's west tower was completed around 1320, it must have attracted widespread awe and attention – nevertheless, the prodigious building did find only few built successors in the following decades. This hiatus can be attributed in part to the fact that the planning and realisation of such an ambitious object required a certain amount of time. Additionally, it has to be taken into account that around the middle of the 14th century crop failures, climate deterioration and plague epidemics caused a noticeable decline in building trade.

Reception in the 14th century

The first subsequent buildings were erected directly in Freiburg, at the minster itself: Tracery spires were added to both of the towers flanking the choir, around the middle of the century. The on-site findings indicate that, at the southern tower, an existing stone structure might have been reworked. For the northern tower, a new openwork spire was realised which repeats the construction of the western tower.[2]
Outside Freiburg, successor buildings were first constructed in the far eastern provinces of the empire. The tower of the pilgrimage church of Maria Straßengel in Carinthia and the "Höckrige Turm" (i.e. "Hunched tower") of Meissen Cathedral. While the Meissen spire was completely reworked after an unsuccessful restoration attempt in the early 20th century, the spire of Straßengel, built around 1365-1370, has been preserved until today. As is clearly visible in the interior of the pilgrimage church's roof, the addition of the tower with its spire was an inspired afterthought. Its general form shows that its design is based on Freiburg, but the execution of profiles and stereotomy indicate that the responsible master builder was obviously not familiar with the Freiburg details; he might have known the tower only vaguely or from drawings. As archaic as the stone structure of the Straßengel tower is formulated, its iron supporting structures are innovatively crafted. Spire and octagon are fitted with ring anchor systems. Additionally, there is an elaborate anchor system above the octagon's vaulting: A large iron ring is placed around the apex of the dome. It consists of two semicircles whose ends are looped into each other. Eight radial anchors are attached to it, they secure the base points of the helmet struts.
The small-scale architecture of the "Schöner Brunnen" in Nuremberg (around 1390) also belongs to the successor buildings of the Freiburg

[2] Regrettably replaced around 1960.

project. In a way, it is a miniaturised model of an openwork tower, with extraordinarily elaborate construction forms that anticipate the large-scale buildings of the following century. The fountain tower, for example, has a multi-storey octagon in which the individual storeys' axis are partly skewed to each other, as well as an elaborate trimming with statues and cathedral gothic elements such as flying buttresses.
Nevertheless, the somewhat exiguous built reception of Freiburg's openwork spire in the 14th century is contrasted by a number of projects that were begun but not completed, or were finished later to modified plans. This is e.g. the case for the enormous project of the twin tower facade of Cologne Cathedral. Its construction plan, the famous "Riss F", was, according to current research (Böker, 2018, pp. 33-35), designed around the middle of the 14th century. Also, the tower projects of the transept tower of St Vitus' Cathedral in Prague, St Stephen's Tower in Vienna or the three-towered Ulm Minster have be mentioned in this context – projects for which presumably large openwork spires were planned from the outset, but for whose design and intended final appearance no plan drawings survive. On the other hand, the magnificent "Regensburger Einturmriss" preserved in the Regensburg diocesan archives is a tower design that has not yet been convincingly assigned to a specific building project. The recently discussed attribution as a possible first draft for the western tower of Ulm Minster remains to be discussed (Böker *et al*., 2011, pp. 31, 36-37).

Reception in the 15th and 16th centuries

After the conspicuously restrained reception of the 14th century, the 15th and early 16th century saw a flowering of the building typology. Starting from its origins in south-western Germany, a remarkable series of openwork spires based on the Freiburg model was erected throughout central Europe, as eventually also in northern Spain **[fig. 4]**. These are:

- 1407-1409: Bebenhausen (D), crossing tower of the abbey church, by lay brother Georg von Salem
- 1410: Bebenhausen (D): Turret on the gable of the abbey's summer refectory, by lay brother Georg von Salem
- ~ 1400-1425(?): Avioth (F): La "Recevresse", a chapel situated at the enclosure of the main pilgrimage church
- 1410: Pforzheim (D): Turret of tthe dominican convent, by Ulrich v. Ensingen, destroyed 1929-1930

- ~1410, Pressburg/Bratislava (SVK): Tower of the Franciscan church
- ~ 1423: Basel (CH), Turret of the dominican church, Hans Kuhn
- 1421-1428: Basel (CH): Northern tower of the cathedral, by Ulrich von Ensingen and Hans Kuhn
- until 1429: Vienna (A); Tower of the collegiate church Maria am Gestade, by Michael Knab and Hans von Prachatitz
- 1427-1433: Heilsbronn (D): Turret of the abbey church, by Hans von Nürnberg
- until 1433: Vienna: Southern tower of the cathedral, by Hans von Prachatitz
- until 1439(?): Strasbourg (F): Northern tower of the cathedral, by Johannes Hültz
- ~ 1444: Kiedrich (D): Turret of St. Michael's chapel, by Peter and Niklas Eseler (?)
- 1449-1455: Brussels (B): Tower of the town hall, by Jan van Ruysbroek
- until 1453: Leuven (B): Tower of the collegiate church St. Gertrude, by Jan van Ruysbroek (?)
- 1442-1458: Burgos (ES): Western towers of the cathedral by, Hans von Köln (Juan de Colonia)
- ~ 1458: Leon (ES): Southern tower of the cathedral, by Joosken von Utrecht
- ~ 1450-1470: Passau (D): Tower of the collegiate church St. Nikola (destroyed 1815)
- until 1468: Leuven (B): Turrets of the town hall, by Mathis de Layens,
- 1445-1482: Esslingen (D): Tower of the Frauenkirche, by Hans Böblinger
- after 1467: Passau (D): Turret ("Stephanstürmchen") of the cathedral by Hans Lindorfer
- 1474: Ulm (D): Calvary, by Hans Böblinger (destroyed 1807)
- 1478-1481: Metz (F): Southern tower of the cathedral, Tour de la Mutte, by Hannes de Ranconval
- ~ 1486: Rottenburg a. Neckar (D): Tower of the parish church St. Martin, by Hans Schwarzacher

- ~ 1475-1500: Rothenburg o.d. Tauber (D): Both towers of the parish church St. Jakob
- 1483 and 1493: Ansbach (D): Side turrets of the collegiate church St. Gumbertus, side turrets, by Heinrich and Martin Echser
- 1496-1500: Basel (CH): Southern tower of the cathedral, by Hans von Nussdorf
- 1479-1503: Meisenheim a. Glan (D): Tower of the Schlosskirche (castle church), by Phillip von Gmünd
- ~ 1509: Notre Dame de l'Èpine (F): Southern tower of the pilgrimage church
- 1506-1516: Thann (F): Tower of the collegiate St. Thibaut, by Remy Faesch from Basel
- until 1519: Bozen (I): Northern tower of the cathedral, by Burkhard Engelberg and Hans Lutz von Schussenried
- 1511-1523: Konstanz (D): Western towers of the cathedral, by Lorenz Reder (destroyed 1855)
- early 16th century: Pfullendorf (D): Tower of the parish church St. Jakob
- 1526-152: Oudenaarde (B): Beffroi of the town hall, Hendrik van Pede
- ~ 1547: Denzlingen (D): Tower of the parish church St. Georg
- ~1550-1551: Arras (F): Beffroi of the town hall, Jacques Lecaron
- until 1551: Oviedo (ES): Southern tower of the cathedral, rebuilt 1576-1587 after lightning stroke, by Juan de Badajoz / Rodrigo Gil de Hontanon
- 1594-1597: Ansbach (D): Central tower of the collegiate church St. Gumbertus, by Gideon Bacher

The defining figure of the first decades of the 14th century was Ulrich von Ensingen, master of the building lodges both of Ulm and Strasbourg. He was obviously well acquainted with the construction methods initiated in Freiburg. Their structural system is repeated in the buildings supervised by him and his successors. This includes not only the adaptation of the profile systems for the octagon halls and the openwork spires but also the expert use of the wrought iron tie rods and the formation of the spires' finials with the typical central iron "skewer".

In his designs, Ulrich pursued the leitmotifs of the Freiburg design and enriched them with his own innovations. A particular concern of his was the

vertical access to the helmet. The high, open stair turret that ascends to the Freiburg tower gallery was, in its time, a highly innovative structure – still, it does not offer any possibility for further ascent above this level. For the towers of Ulm, Strasbourg and Esslingen, Ulrich therefore envisaged an additional stair turret to be built inside the spire ("spire staircase"). These turrets should rest on the apex of a vault dividing spire and octagon hall. At the top of the helmet, a stone "crow's nest" was to provide the exit. Likewise, Ulrich conceived inwardly curved, concave helmet contours, this to a certain extent a counter-design to the apparently convexly curved Freiburg helmet.
During his lifetime, Ulrich only saw the completion of the small turret of the gable of the Dominicans' convent church in Pforzheim. The north tower of Basel Minster was finished a few years after the master's death by his son-in-law Hans Kuhn; here, for the first time, a concave contour could be realised. The spire of Strasbourg Cathedral was built by 1439 by Ulrich's successor Hans Hültz according to a divergent design, and the spire of Ulm Minster spire could not be completed until the 19th century. At least the tower of Esslingen's Frauenkirche ("Our Lady's church") was realised in the late 15th century by Hans Böblinger, following on Ulrich's daring concept with a stair turret in the spire's interior.
Although Ulrich's designs were only partially realised, his influence manifests decisively in the tower projects in the west of the empire. For example, the first spire stair staircase to be realised can be found on Jan van Ruysbroek's tower of Brussels town hall tower (1449-1455) (Kayser, 2018a), which was based on Ulrich's design for Strasbourg. Further spire staircases were implemented on the Tour de la Mutte of Metz Cathedral and at Meisenheim Castle Church.

The spread of the typology in the 15th century can be traced along the two major river systems of Central Europe, the Rhine and the Danube, with Freiburg roughly at the intersection of the two transport axes. While the tower projects in the south west and along the Rhine stood under the influence of Ulrich von Ensingen and his successors, an independent group of openwork towers was developed in Vienna, by and in the periphery of the important building lodge of St. Stephen's. Its master, Peter von Prachatitz, not only created the slender spire of St. Stephen's Tower around 1425-1435, but was probably also responsible for the idiosyncratic domed spire of the Gestadekirche ("River bank church") (Hassmann, 2002, pp. 227-228). Both buildings exhibit a specific layout as well as characteristic profiles and construction details. The Viennese building designs finally ra-

diated westwards again, where in Passau, the seat of the bishopric initially responsible for Vienna, a large openwork tower was built at the collegiate church of St. Nikola (unfortunately destroyed in the early 19th century) as well as the sophisticated cabinet piece of the "Stefanstürmchen" on the cathedral. The latter copies the domed design of Vienna's Gestadekirche.
A fascinating case of transfer of form and structure can be found in northern Spain, where large openwork spires were erected at the cathedrals of Burgos, Leon and Oviedo. The path of transfer is astonishingly well documented: Bishop Alonso de Cartagena, of the see of Burgos, visited the Council of Basel around 1440. There, he admired Ulrich's spire at the cathedral and eventually also travelled to Strasbourg and nearby Freiburg (Menéndez, 2016, pp. 26-28). The bishop succeeded in convincing Hans, a stonemason from Cologne, to relocate to Spain, where he completed the wester twin towers of Burgos cathedral with two large openwork spires. Under the hispanicised version of his name – Juan de Colonia – Hans became the founder of a family of master builders. The Burgalese spires served, in turn, as a model for the southern tower of the cathedral in Leon around 1475-1480, and from there the moulds reached Oviedo around the middle of the 16th century.
The Oviedo spire had to be reworked around 1575 after being struck by lightning; one of the last great examples of the typology was created. Fascinatingly, this final spire, built by Rodrigo Gil de Hontanon, directly refers back to the 250-year-old design of Freiburg's west tower: During the reconstruction, an octagonal hall, originally missing, was added, which opens up to the spire without a separating vault. The profiles of the hall's pillars as well as of the corner struts stand in tradition of tower projects influenced by Ulrich von Ensingen. Its four stair turrets accompanying the octagon presumably also hark back to Ulrich's design for Strasbourg Cathedral. Gil de Hontanon proved to be closely familiar with the tower buildings of the German southwest. Thus, his Oviedo design impressively testifies to the outstanding role of Freiburg's west tower as reference object of a building typology that had grown far beyond the regional context.

Construction technique

Construction systems

The phenotypical similarity of the various medieval spires belies the fact that tracery pyramids can be constructed in different ways. Following the chronology of development, the following variants can be named **[fig. 5]**:

a) Insert tracery
b) Continuous circumferential hull
c) Elemented hull (“Freiburg system”)
d) Grid insert system (“Cologne system”)

a) Insert tracery

With regard to the chronological development, this is the earliest form that preceded the actual blossoming of the typology in the wake of the Freiburg construction.

Basically, the construction follows the design principles of window tracery (Kayser, 2012, pp. 87-90). Openings are cut into the closed, sloping surfaces of the spire’s flanks, into which independent small “tracery windows” are inserted. The internal structure is independent of the surrounding masonry and designed as a separate insert element. Strictly speaking, the spires executed according to this construction variant are still “conventional” stone tower pyramids whose isolated recesses are filled with decorative elements.

The construction method was not abandoned even after the Freiburg innovations, and can still be found in buildings of the 14th and 15th centuries: The structure of the monumental spire of Cambrai Cathedral, created around 1360 and collapsed in 1809 (Thiébaut, 2015, pp. 433-442), as well as the spire in Rottenburg am Neckar, built around 1460-1470 by Hans Schwarzacher, were built in accordance with this system.

b) Continuous circumferential hull

Chronologically, this type of construction stands at the origin: The first, lowest storey of Freiburg’s spire, and, thus, the first attempt at realising an openwork system, was executed according to this plan.

There is no differentiation of the stone hull into tracery panels and struts. Instead, a circumferential joint section is developed for the entire storey; the openwork spire forms a kind of “3D puzzle”. No distinction is made between the supporting system and the fillings; the tracery itself forms the load-bearing skeleton of the spire.

The system affords considerable efforts in design, production and execution. Dimensional tolerances between the individual workpieces must be avoided, as the precisely interlocking overall structure might easily be disturbed by small local errors. The concept demands an outstanding geometric understanding from the designing master: Both the overall

structure and the complex individual stereotomy of the single workpiece have to be determined precisely before construction commences.
With regard to the considerable challenges in planning and realisation, it is downright surprising that any spires were realised according to this construction principle at all – this has to be read as a deliberate display of craftsmanship and architectonical refinement. Particularly elaborate examples are the spire of Maria am Gestade in Vienna (around 1420-1430) and the spire of the Frauenkirche in Esslingen (before 1480).

c) Elemented hull (Freiburg system)

The system, developed during the construction of the Freiburg spire, formed the reference for medieval openwork spires. It combines a consistent architectural design with an economically feasible construction system.
In this system, again, the hull construction is made entirely of tracery sections from which, however, the corner struts are separated as "joints". This results in the spire's flanks being divided into rectangular (trapezoidal) individual panels ("tracery panels"). On smaller turrets, these can be monolithic, but on a large spire, such as the towers of Freiburg or Vienna, the panels are composed of several individual workpieces. Wedge-shaped struts are inserted between the rectangular panels at the corners. These mediate geometrically between the pyramids' flanks, whose edges could thus be worked with a simple, orthogonal joint formation. Independent crocket elements could be placed on the extrados of the struts as ornamentation. The elementation of the flanks into individual rectangular panels results in horizontal circumferential caesurae, which divide the spires into horizontal storeys. In these horizontal joints, wrought-iron ring armatures could be integrated to reinforce and stabilise the stone skeleton. The joints between the individual anchor sections are set in the corner stones of the struts.
In the case of very small tower spires, it was even possible to dispense with independent corner struts and to butt the tracery sections at the corners with a mitre – however, this was only realised in the case of very small tracery spires, such as on the little bell tower of the Bebenhausen refectory (around 1410).

d) Grid insert system (Cologne model)

The construction system is an alternative to the Freiburg model. Basically, it is derived from the early "Insert tracery". Unlike both systems developed

and implemented in Freiburg, the tracery does not form the load-bearing hull of the spire; here, the structure is differentiated into load-bearing elements and inserted fillings. Instead of closed masonry surfaces forming the hull, there is an orthogonal "spatial grid" of inclined corner struts and horizontal stone crossbeams (traverses). Between the traverses and the struts, square open spaces were created, tapering trapezoidal towards the top, into which the tracery panels could be inserted. The spire is thus divided into a sequence of horizontal storeys. Phenomenologically, the resulting structure resembles the Freiburg tracery pyramid, although the construction on which it is based differs considerably in concept.
Probably the earliest example of this system is found in the famous Riss F of Cologne Cathedral.[5] The traverses between the tracery panels are clearly visible on the plan and were probably intended to be as strong as the struts. As is well known, the design could not be realised in Cologne – or only after a delay of several centuries. However, a culturally and historically remarkable export was achieved: The Strasbourg spire of master builder Hans Hültz, called from Cologne to Strasbourg, was built according to the "Cologne system". The finely articulated tracery of the pyramid is inserted between the enormous struts, which also support the spire stairs, and strong horizontal crossbeams. The actual construction of the tracery pyramid is "masked" by the rich decoration of the spire's exterior with stairs and pinnacles.

Structure and load transfer[3]

The load transfer on a Gothic openwork spire is not intuitively comprehensible. In the course of the restoration of the largest surviving stone tower spire in Freiburg 2009-2019, a comprehensive structural analysis had to be carried out in order to analyse the cause of cracks and fissures in the stones and to be able to recommend a suitable safeguarding strategy. The analysis' results can be transferred to smaller and less elaborately joined structures. Three models with differentiated levels of detail were used for the computational representation of the load-bearing behaviour, which provide almost identical quantitative results:
From a static point of view, the tracery panels cannot be considered as

[3] For details Barthel *et al.*, 2017; Barthel, Tutsch, Jordan, 2016; Barthel *et al.*, 2014.

continuous planes with recesses. The panels consist of individual workpieces and the connections between them can only transmit compressive forces as well as and friction. The load transfer occurs as a series of additions and resolutions of forces; the global load-bearing effect can be described in a simplified way as a spatial framework of compression struts (stone elements) and tensile elements (wrought-iron bars).
The dead weight of the stone structure is transferred via the struts and via the “truss” of the tracery panels. Dependent on their individual design, local tensile stress may develop as a result of force resolutions; as consequence, cracks and fissures occur in the stone. Fortunately, the stone structure is so “tightly” woven that a damage-free load path for the dead weight can be found in the tracery. Globally, the eight faces of the spire lean against each other as a result of their inclination. In addition to the forces flowing down along the struts and tracery, there are also horizontal compressive forces. These are not very large due to the steep inclination of the tower; they are also superimposed by forces from other effects, which will be described below. The forces running downwards along the struts and the tracery have to be deflected vertically at the base of the pyramidal spire. A ring anchor is therefore required at this position.
The critical load – without considering the special case of earthquake – is the wind load. Naturally, there was no normative basis for the wind load on a perforated pyramid. Therefore, wind tunnel tests were arranged to determine the resulting wind force as well as the height and circumferential distribution forces.[4] For this purpose, a 1:100 model and a separate 1:40 model of separate storeys of the spire were created with a 3D plotter. In addition to the quantitative wind forces (normative 50-year wind for the Freiburg site), which were directly incorporated into the structural calculations, the results provided two fundamental findings, scientifically documented for the first time: Contrary to the intuitive assessment that an open cross-section has a lower wind resistance, the resulting wind load is almost twice as high as with a comparable closed octagonal pyramid. In addition, significant tangential wind forces have to be considered, especially on the wind-parallel surfaces/flanks. This effect stems from the “roughnesss” of the tracery profiles.
A main result of the calculations for the load case wind and dead weight is

[4] Carried out by Wacker Ingenieure, Birkenfeld.

that the tensile effect of the wrought-iron anchors is necessary for stability. The load transfer can be compared in a simplified way to a truss structure. Diagonally running compression struts are formed within the tracery fields – as far as the tracery forms allow. The iron anchors have the task of "tying back" the forces. Thus, tensile forces have to be transferred from one iron bar to the next iron bar at joint in the cornerstones. As a result of the change in direction, a force component directed into the center of the spire is created, which hast to be absorbed by the stone surrounding the joint.

Use of wrought iron elements

As the structural analysis shows, the integration of wrought-iron elements is essential for the structural integrity of the wind-exposed and vulnerable openwork spires (cf. Kayser, 2018b). Here, the traditional image of the Gothic as a "pure" stone construction, in which only compressive forces are transferred via the stone skeleton of vaults, arches and pillars, must be corrected – iron elements were expertly used to absorb tensile forces in the structure, so that in some cases it is almost appropriate to speak of a "reinforced" construction.
Iron elements were used in Freiburg for the horizontal ring ties in the tracery fields of the stone pyramid. Likewise, the base of the spire as well as the arcades of the octagonal storey are secured with corresponding circumferential anchors. Finally, the construction of the characteristic finial, which forms the basis of the spire, was only possible through the use of iron "reinforcement" **[fig. 6]**. A static analysis asserts that its elements are remarkably well adjusted to stresses by vertical dead load as well as wind loads: The top end of the central wrought iron skewer is fitted with an iron cuff, thus fixating the stone elements threaded onto the vertical bar. In the case of horizontal loads onto the pinnacle, the iron skewer, can accordingly take up tensile forces, and the stone elements on the lee side are charged with compressive forces.
At the Freiburg spire with its ring anchors in every tracery storey, 4 tons of wrought iron were used, in comparison to roughly 620 tons of stone – for the conditions of the early fourteenth century an astonishing amount of the expensive material **[fig. 7]**! Presumably, the implementation of the iron armatures into the spire was only possible due to the highly developed "steel and coal" industry of the nearby Black Forest.
With the takeover of the Freiburg building forms and constructions, the

iron elements were also adapted. The anchor systems were mostly integrated into the stone structure, as in the model. Ring anchor systems can be found in the vicinity of Freiburg on the Upper Rhine on the spire of the Basel Minster, on the spire of the Theobald Minster in Alsace or on the parish church of Denzlingen, but also on the distant town hall spire of Brussels or the cathedral spires of Burgos. Tower pinnacles modelled on the Freiburg finial have also been detected in the course of recent building investigations, for example on the finial of the Cistercian monastery church of Bebenhausen (Kayser, Tutsch, 2015), or on the spire of the St Stephen's tower in Vienna (Kayser, 2019). According to historical plans of the spire, which was unfortunately demolished and reconstructed in 1892-41, this tower had a particularly elaborate structure: Due to the steep inclination of the Vienna spire, the pinnacle has a total height of nearly 12 metres. From the surviving accounts we know that the skewer was forged to a length of 20 m(!). The name of the master of the forges is known: A blacksmith called Andres, "Schmied beim Stubentor" was responsible for the works. The Vienna skewer extends app. 8 m into the open interior of the spire pyramid. Its bottom end was fixated with a strong horizontal iron bar. In the early 15th century, Ulrich von Ensingen as well as his descendants and students played an important role in the dissemination of the building technique. They occupied leading positions in the building lodges in south-western Germany and the neighbouring regions (Alsace, Switzerland), thus generating a close network of mutual exchange. An original plan drawing of Ulrich or his son **[fig. 8]**, Matthäus Ensinger, for the lower section of the Ulm tower illustrates the astonishing ability of the Ensingers for integrating iron elements into the 'stone skeleton' (Cf. Böker *et al.*, 2011, pp. 43-54). This unique document shows an elaborate iron armature, with the typical joints consisting of iron rings and forged bars with crooked ends in the masonry massifs of the tower's corners. The design was, in fact, executed as planned. Similar systems can also be found on other buildings overseen by Ulrich, e.g. at the octagonal section of Strasbourg cathedral tower.

Résumé

Following the model of the Freiburg tower, about 40 subsequent buildings of different dimensions were built between the middle of the 14th and the middle of the 16th century. The main hub of the architectural reception was in central Europe, along the two great river systems of Danube and

Rhine up as far as to the Netherlands, but travelling master builders also brought the typology to northern Spain and South Tyrol - at Freiburg, a European building typology originated that shaped medieval sacred architecture across countries.

The spire on the west tower of Freiburg Minster and its medieval successors are not only highlights of medieval architecture in terms of volume and formal effort, they are also outstanding monuments of the construction history. Stone tracery, originally an architectonic element from a different context, was adapted for a new building task. For its application and implementation on stone spires – the "skyscrapers of new Jerusalem" (Bork, 2003) – efficient structural systems were created. With the integration of wrought iron armature, the hybrid construction of "reinforced stone" was and is able to permanently withstand considerable forces.

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Fig. 1. Freiburger Münster, view from south, with its three openwork towers.

Fig. 2. View from the octagonal hall into the stone pyramid of the Freiburg spire.

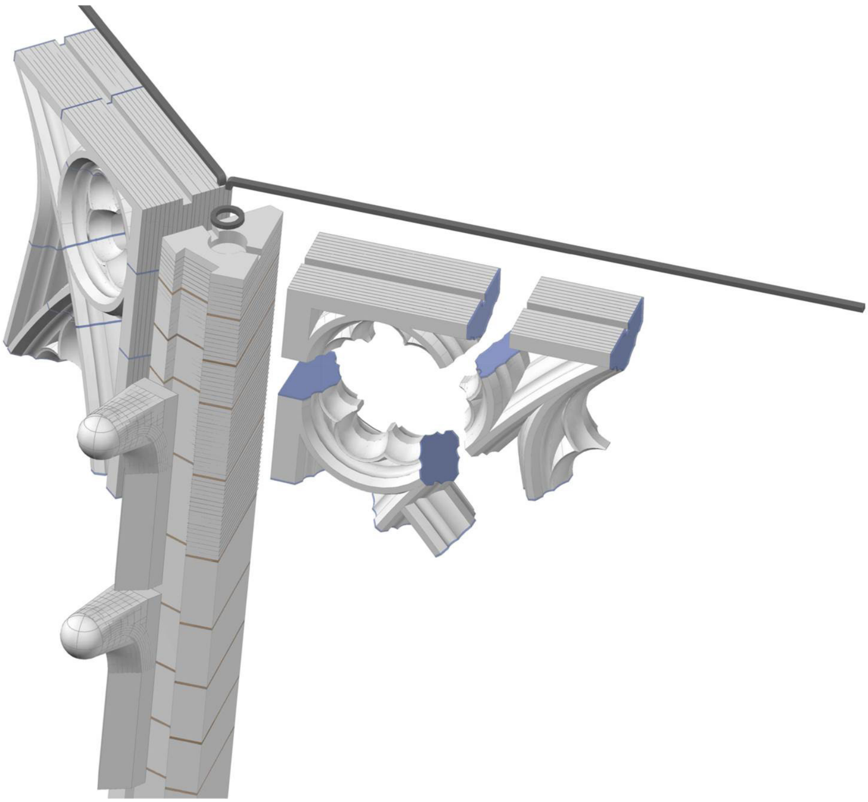

Fig. 3. Construction system of the Freiburg spire, with tracery panels, corner struts and wrought iron armature.

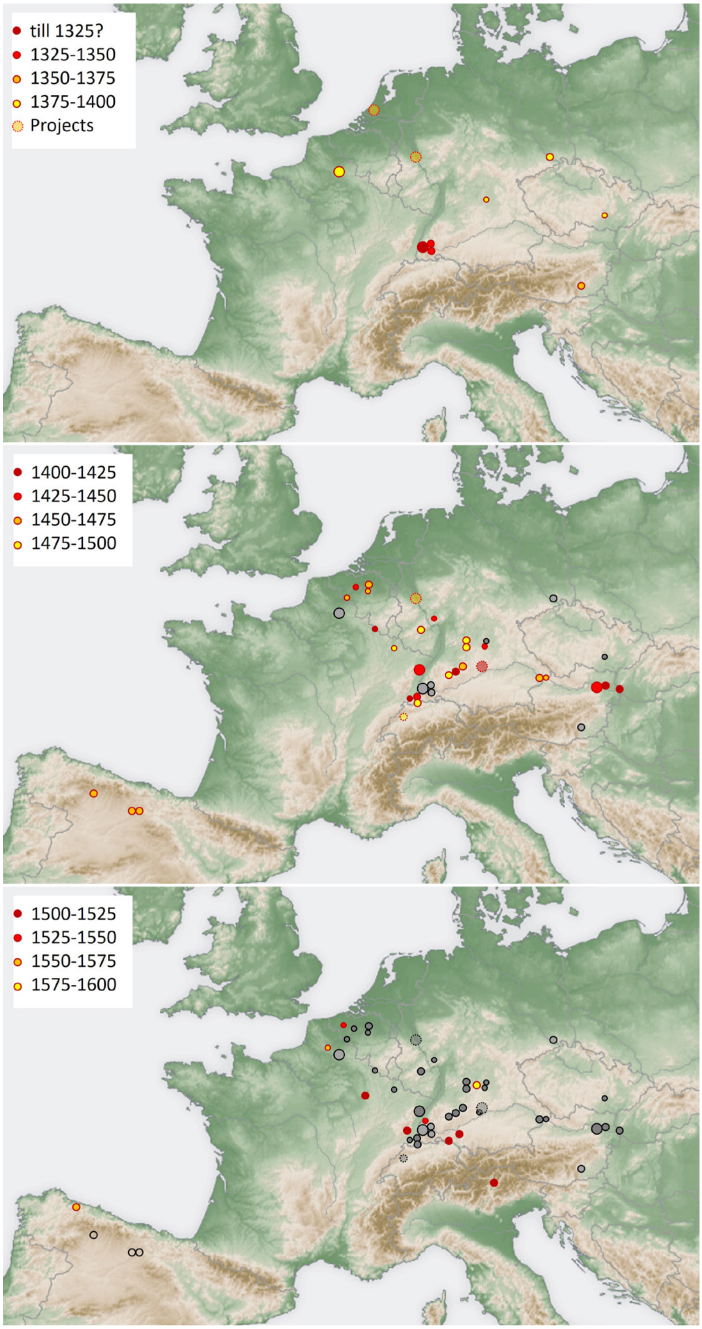

Fig. 4. The dissemination of the openwork spire in the 14th century (top), the 15th centure (middle), 16th century (bottom). (Author, on the base of wikipedia maps).

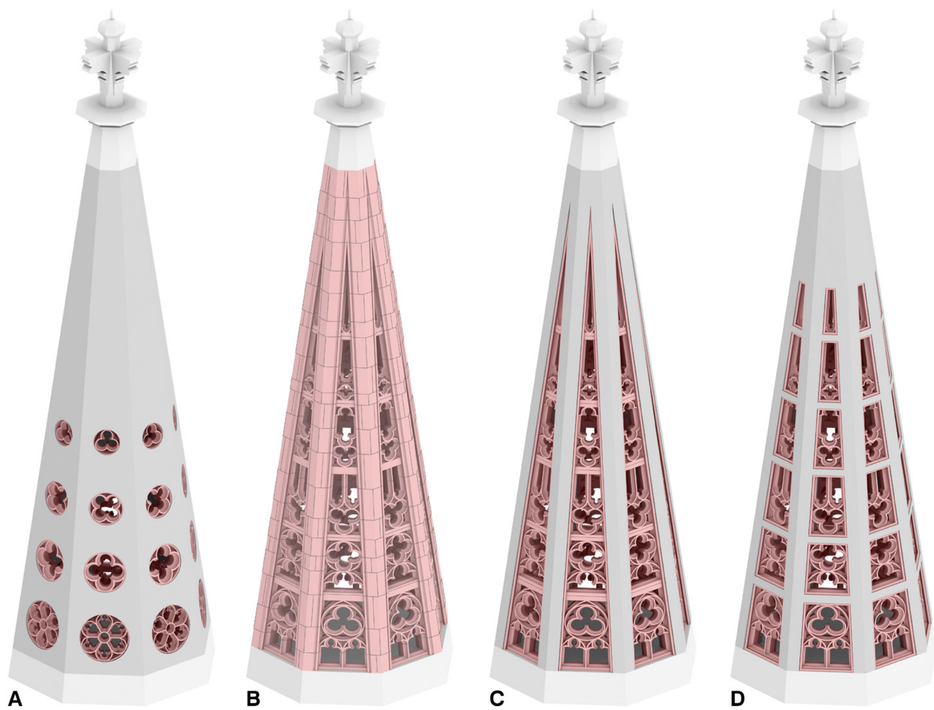

Fig. 5. Construction systems for openwork spires. From left to right: a) Insert tracery, b) Continuous circumferential hull; c) Elemented hull ("Freiburg system"); d) Grid insert system ("Cologne system").

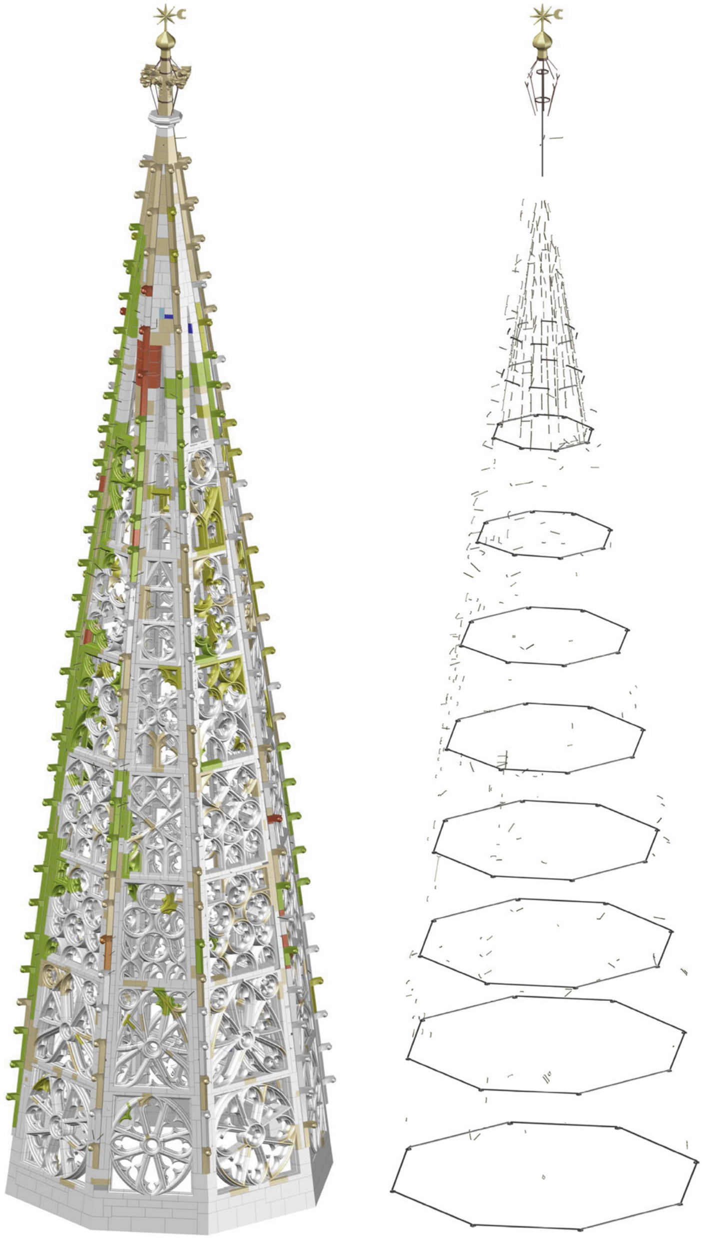

Fig. 6. Virtual model of the Freiburg spire. Left side: Stone structure, right side. Wrought iron elements (Barthel & Maus GmbH, Munich, 2013).

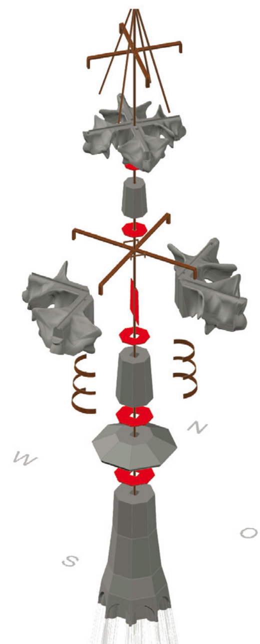

Fig. 7. The pinnacle of the Bebenhausen spire (Kayser & Tutsch 2015), with a central wrought iron skewer.

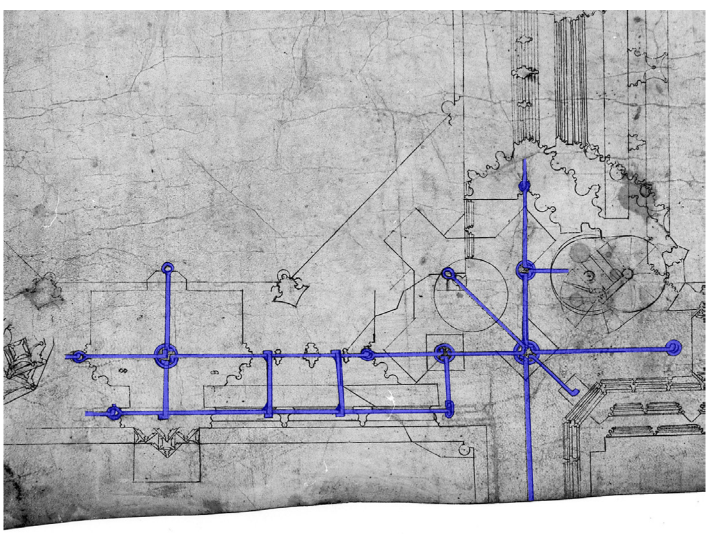

Fig. 8. Plan of Ulrich or Matthäus Ensinger for the western tower of Ulm, with iron armature (Archiv Münsterbauhütte Ulm).

Barbara Berger

Gasholder construction. A multi-disciplinary history*

I. *Introduction*

Long before the rising structures of gasholders changed the cityscapes, it was the gaslight itself that was revolutionizing daily life in the cities at the beginning of the 19th century[1]: towards the end of 1813 public illumination from gas was introduced for the very first time in Westminster, London.

Gasholders are technical buildings that were constructed to store locally produced coal gas used for lighting in the 19th and early 20th centuries. These emerging iron structures presented a new kind of industrial architecture and became a symbol for the gas industry.

The function of the gasholder determined its structure and was initially built with a water-sealed system composed of a water tank, a guide frame and lifts. During the 19th century the gasholder advanced from the bell and telescope-type (linear and spiral-guided), working with a water-sealed system, to the waterless or dry gasholders (piston-type gasholders). Thus a new building type emerged, and was the result of the interaction of various disciplines: iron and gas technology, structural engineering and architecture.

In the 1960s coal gas was gradually replaced by natural gas and new methods of storage were used. Today, the surviving historic gasholders are industrial relics: many of them had been already demolished.

This paper summarises the centenary-long history of the gasholder, its

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[1] This research was limited by gasholders used for public gas lighting only.

function, development, form and structure as a part of engineering and industrial heritage[2].

II. *Gaslight – ex Fumo Dare Lucem*[3]

It was in 1807 that the German merchant Frederick Winsor (1763-1830) first demonstrated gas lighting in Britain when he lit part of Pall Mall in London. In 1813, the first public gasworks were built at Westminster by the engineer Samuel Clegg (1781-1861)[4] and on 31st December 1813 the illumination of the Westminster Bridge was inaugurated (Clegg, 1841). This was the very first public illumination with gas – the beginning of a new era – and London was its nucleus[5].

With the invention of coal gas, the need arose for a suitable gas storage reservoir. In 1815, the German chemist Friedrich Christian Accum (1769-1838) wrote the first treatise about gaslight, its manufacture and the technical apparatus required (Accum, 1815). The great interest in this new technology is reflected in the early translations from English into German and French in 1816 and into Italian in 1817.

The production of lighting gas involved a complex range of different machinery and pipes through which the gas had to pass before it achieved a good illuminating quality and was stored in gasholders. The production of coal gas at the end of the 19th century was divided in three phases:

1. Distillation of coal in the retorts (decomposition of coal), which started by reaching a temperature of 1000°C.
2. Physical purification of the raw gas and cooling separate out the liquid bi-products by condensation[6].

[2] This paper is based on the dissertation "Der Gasbehälter als Bautypus", published in 2019 by Barbara Berger.

[3] LIGHT FROM SMOKE – citation by Horace used by Accum in his treatise.

[4] Clegg was very empirical in his search for suitable designs of gasholders on the site of Westminster, whose ground conditions were difficult; besides the design derived by Lavoisier, he experimented with rotating and collapsing gasholders (Accum, 1819).

[5] The new lighting technology with gaslight spread rapidly outside the United Kingdom, to France (Paris, 1819), Germany (Hannover, 1825) and Italy (Turin, 1837).

[6] At this stage, the gas consisted of a dense brown vapour due to the volatile, liquid and

3. Chemical purification.

Finally, the volume of gas was measured by using a meter, the so-called *gasometer,* and then introduced into a storage building – the *gasholder* – through an underground inlet-pipe. The gas was fed from the gasholder through the outlet-pipe and a device to regulate the pressure and then into the public distribution network.

III. *Function and categories*

The gasholder`s structure was determined by its function and the need to fulfil two basic requirements: a variable capacity, and a gas tight construction.
A water-based system met both requirements. It was composed of two primary elements **[fig. 2]**: a water tank and a receptacle for the gas. The latter was a kind of vessel, closed at the top and open at the bottom. Reflecting its form it was often called the *bell*. It was immersed in the tank and rose and fell according to the current volume of gas. This method of operation also served as an indicator: if the lift was not visible, the gasholder was empty; if it was at maximum height, it was obviously full. It was due to this movement that the vessel acquired its common name *lift*.

1. *Bell-type gasholder (single-lift gasholder)*

This type had a cylindrical ground plan and was composed of a water tank, a single lift and an external guide frame which guaranteed the reliable movement of the lift. Vertical guide rails were mounted on each column or standard, along which the lift`s rollers ran up and down.
The predecessor of the cylindrical gasholder was a very small rectangular apparatus developed in the 1780s by the French chemist Antoine Laurent de Lavoisier (1743-1794) forming part of his "Appareil du souflet hydrostatique". He used for experiments with oxygen and called this apparatus "Gazomètre" (Lavoisier, 1782; Lavoisier, 1789b).
Clegg adopted Lavoisier's principle for his first gasholders for public illumi-

solid by-products (coke, tar, ammonia, hydrogen sulphide, carbon disulphide, cyanides) (Accum, 1815; Calzavara, 1899).

nation **[fig. 1]**[7]. They had a small capacity, as they were primarily intended to supply the main squares and streets of towns.
In 1890, the bell-type was developed to incorporate helical guides which avoided the need for an expensive guide frame (see chapter III.2.a).

2. *Telescope-type gasholder (multi-lift gasholder)*

Due to enthusiastic adoption of gas lighting, greater storage capacity in gasholders was needed. A system was developed with a second telescopic which almost doubled the capacity of the gasholder **[fig. 2]**.
In 1818, William Stratton (dates unknown) registered the first patent for a "doubled gasholder" with a telescopic system[8]. The first telescopic gasholder was successfully erected in 1835 by Stephen Hutchison (dates unknown) at a gasworks in Vauxhall, London (Clovis, 1836).
As the original bell-type gasholder was extended into a two-lift or telescopic gasholder, so a second level or tier was also added to the guide frame. The most complicated feature was the gas-tight seal of these lifts. When the inner lift was full of gas, it left the tank with its water-filled cup and engaged with the dip of the second lift. The water-filled cup ensured the gas-tight device. This telescopic system was further developed into multi-lift gasholders.
The largest gasholder of Great Britain was built in 1892 with six lifts (v= 339,802m3) in East Greenwich, London **[fig. 6-E]**.

a) Frameless gasholder

In 1890, "The Journal of Gas Lighting" announced a "Revolution in Gasholder Construction" by William Gadd (1838-1919) and William Mason (1849-1918), who, in 1887, had invented a new frameless guide system for the movement of the lifts (anon., 1890; Elton, 2014), which radically changed the appearance of the telescopic systems: **the spiral-guided gasholder [fig. 2]**.

[7] William Murdock, together with his apprentice Samuel Clegg, had already built a small gasworks in 1792 at the Boulton & Watt cotton mill in Soho. It had a cylindrical bell-type gasholder (Clegg, 1841).

[8] Another concept was shown by William Tait and his invention "gazomètre à lunettes" (see Newton, 1825).

The gas seal worked using the same principal – cups and dips. But the lifts rose and fell by twisting, one lift inside the other and they no longer needed an external guide frame. This reduced construction time, materials and costs. Inclined guide rails were fixed directly onto the surface of the iron shell. The lifts moved up and down like a screw: each lift going in the same direction or rising alternately clockwise and anti-clockwise. The spiral-guided gasholder had a fascinating and very characteristic mechanism, which allowed gasholders with an underground tank almost to disappear when empty.
The first Gadd & Mason gasholder (v= 3,086m3) was realised 1889 in Northwich, Cheshire by the engineer Thomas Newbigging (1833-1914). It had two lifts, that rose in the same direction and had internally mounted helical guide rails (Anonymus, 1890).
In the same year another frameless technique was invented by Edward Pease (1861-1934): **the rope-guided gasholder**. However, its reliability and durability was much lower than the spiral-guided system (Pease, 1896).

b) Flying lift

The frameless concept led to a further development by George Thomas Livesey (1834-1908)[9] and his brother Frank (1844-1899). In 1887 they successfully realised a third **flying lift** on a two-lift gasholder ($h_{per\ lift}$= 7,62m) for the first time in Rotherhithe, London (Livesey, 1888)[10]. This allowed another lift to be easily attached to existing gasholders, without enlarging the guide frame. These flying lifts **[fig. 6-E]** rose over the upper edge of the guide frame without external support. This technique also used the spiral-guided system. In the case of the gasholder in Rotherhithe the guide frame was reinforced with diagonals into a braced structure.
Hydraulic gasholders in the 19th century worked on a low-pressure system (up to 125mm water, 12,5mbar) (Calzavara, 1899).

[9] G.T. Livesey was an important pioneer and gas-engineer working in London (South Metropolitan Gas Company) and was awarded the title “Sir” in 1902 for his inventions and contributions to the construction of gasholders (Chrimes, 2008). Further information on Livesey’s life and opera, see Mills, 2013.

[10] Their design was based on an 1851 patent by Joshua Horton, which described the attachment of small posts on the dip of the second lift, in order to support the free flying lift. However, this design did not stand the test of implementation (Dingler, Dingler, 1852).

3. *Piston-type gasholder*

At the end of the 19th century the water-sealed gasholders reached their construction limits: the dimensions of the water tank became larger and, with it, the quantity of water and the ground pressure increased. Furthermore, in winter periods the huge water basins had to be heated to prevent them from freezing.

A new building technique solved all these problems at the beginning of the 20th century: the waterless or dry-sealed gasholder. In 1913, the German engineer Konrad Jagschitz (1876-1941) from **Maschinenfabrik Augsburg-Nürnberg (MAN)** invented the piston-type gasholder, that used a dry-sealed system (MAN, 1913) **[fig. 2]**[11].

This new type consisted of a polygonal closed structure and a built-in piston, that moved up and down depending on the current gas-content. Rollers were fixed to the top and bottom of the framework of the piston. The sealing technique used with a fluid that continuously ran down the shell, was collected at the bottom and then pumped up again[12]. A cup was attached to the edge of the piston, which held a certain amount of sealing fluid and ensured gas-tight seal between the piston and the shell.

Another type was patented in 1926 by the company of the engineer **August Klönne, Dortmund** (1849-1908) which used a rubber-sealing technique: rubber blocks, that were lubricated with grease, and were fixed along the rim of the piston. But this technique was not as reliable as the MAN-type, so that the latter prevailed on the market (Klönne, 1926).

Bell, -telescope- and piston-type represented the main types of gasholders in terms of their technology and each type has its own subgroups determined by the development of the construction.

IV. *Construction history*

The water-sealed gasholder and its distinct construction elements underwent a remarkable technical and structural development, which resulted in the various forms of lifts, guide frames and water tanks.

[11] Jagschitz received an honorary doctorate for this revolutionary invention.

[12] Originally tar was used as a sealing fluid; later it was substituted by mineral oil.

1. *Lift*

The key elements of the lift were based on a cylinder: a flat roof or crown, cups and dips along the upper and lower rims, internal uprights and rollers. The lift was covered with riveted single plates of wrought-iron.
Apart from the complicated construction of cup and dip, it was the roof or crown, that showed the most evident advancement.

a) Flat roof

The upper lift initially had a flat roof **[fig. 3]** with no girders or trusses supporting the cover. The connection between the horizontal and vertical iron plate was therefore crucial and required a strong upper ring beam to transfer the resulting large loads. The gas supported the roof when the holder was full. When empty, a rest frame was needed to support the entire surface. This showed an early example, built in 1829 in Fulham, London, which had a flat roof (d= 15,20m; h= 5,50m) and even some diagonal braces. (Newbigging, Fewtrell, 1879)

b) Flat crown – trussed

To reduce the large horizontal loads, the flat crown **[fig. 3]** was introduced and almost completely replaced the flat roof in the middle of the 19th century. Several radial trusses supported the crown and were united by a central *flying post*. When empty, this construction needed only a central king post or a frustum. Yet, the upper ring beam remained an important detail needed to provide a reliable connection between the crown and shell. The trussed crown was a very common design and was implemented in a two-lift gasholder at Bromley-by-Bow, London (d= 60,19m; rise= 2,74m), that was built in 1876 (Newbigging, Fewtrell, 1879).

c) Flat crown – untrussed

Another widely used flat crown was the untrussed form **[fig. 3]**, that followed the same principle as the flat roof, with no girders or trusses supporting the crown. Thus, a rest frame was required, often in combination with a frustum-shaped tank.
Samuel Clegg Jnr. described in 1841 a single-lift gasholder (v= 4,250m3) with an untrussed crown (d= 26,67m), which when empty was supported by small posts attached on the frustum of the tank.

d) Flat crown – ribbed

The ribbed crown **[fig. 3]** was a special form invented and mainly built by Vitruvius Wyatt (1824-1897): the supporting structure was reduced to radial and annular beams riveted to the iron plate of the crown. This meant that construction elements no longer reached into the interior of the lift and only one central king post was required when empty. In 1874, the first ribbed crown (d= 44,92m; rise= 2,20m) was realised at Redheugh for a two-lift gasholder (v= 28,317m3) (Newbigging, Fewtrell, 1879).

e) Flat crown – lattice

In the meantime, in Germany, the engineer Johann Wilhelm Schwedler (1823-1894) refined Wyatt's principle with its advantage of non-projecting elements in the lift, by means of stiffening **[fig. 3]**. In 1863, he rebuilt the crown of a gasholder building in Holzmarktstraße, Berlin with a lattice dome (d= 30,90m; rise= 3,96m) with cross-bracings in each bay. This was now a self-supporting grid shell and it was no longer necessary to provide a rest frame in the empty state (Schwedler, 1866).
The first Schwedler-type flat crown of a lift was erected in Fichtestraße, Berlin in 1876. The two-lift enclosed gasholder (v= 30,000m3) had a lattice crown (d= 50,72m; rise= 4,00m) on its upper lift as well as on the enclosing structure (Anonymus, 1876).

2. *Guide frame*

The purpose of the guide frame was to ensure linear movement of the lift by means of an attached or integrated guide rail. Following the progress in iron technology and the increasing demand for gas, the guide frame – the fixed, visible part of the gasholder – underwent remarkable developments **[fig. 4]**:

a) Column structure

The early guide frames for bell-type gasholders consisted of free-standing single masonry columns or tripod standards of cast-iron[13], which had no

[13] Another early type implemented timber posts, that were used for the column structure.

horizontal connection. This column structure was often used in combination with a fully underground tank. Most of such column structures were used for single-lift gasholders: only rarely for two-lift ones.
The unique column structure of a small gasholder (v= 5,663m3) still survives in Fulham, London with 12 cast-iron tripod standards (h= 9,14m), erected in 1830 to the plans of Kirkham (Tucker, 2014).

b) Framed structure

With the advent of the telescopic system and the increasing capacity of the lifts, larger guide frames were also required to prevent overturning. This was initially the case with Hutchison's telescopic gasholder (1834), whose four columns were connected with diagonal girders (Clovis, 1836). However, the polygonal composition of the girders prevailed, because it provided a more stable structure.
Early frames were usually built with cast-iron columns[14] and later of various kinds of riveted wrought-iron standards (T-, I-, box-section). Bell-type gasholders were mostly fitted with a single tier – **low single order**[15]. Initially, the columns of early telescopic gasholders were simply extended to the **tall single order**. Occasionally a pair of columns was even used in each corner to reinforce the tall, slender column, or additional intermediate girders were added. Soon, however, a more stable multi-tier system was adopted: for each additional lift, another tier was added. This frame-type thus followed the **additive order** and, until the second half of the 19th century, served as an indicator for the number of lifts. A very typical Victorian style guide frame by Joseph Clark (dates unknown) was built in 1866 in Bethnal Green, London. The two-lift gasholder (v= 25,485m3) had two tiers, each of 16 tapering columns **[figg. 6-B, 7]** (Tucker, 2014).

c) Braced structure

A further reinforcement of the guide frame was the diagonal bracing in

[14] More rarely, gasholders were built with standards of cast iron – such as the diamond-shaped ones by Wyatt in Redheugh (ca.1874) Fulham, Kensal Green or Beckton (from 1879) (Newbigging, Fewtrell, 1879).

[15] These early designs also had to guide the lift. They were also fitted with rollers for the chains of the counterweights, that were attached to the top of the lifts (early lift constructions generated very high stresses due to their weight).

each bay. The **single cross bracing** was initially simply added to the usual frame structures, as William Mann (dates unknown) did in 1866 in a three-tiered guide frame (h= 32,92m) with 12 cast iron columns in each tier (Newbigging, Fewtrell, 1879).
In 1870, a new era of guide frames began: gas-engineers moved from empirical design to detailed calculations (Milbourne, 1923)[16].
After bracing with a single cross, it was George Livesey again, who invented a ground-breaking structure – with a **double cross bracing [fig. 4-C]**. He not only inserted this double cross in each bay, but fixed the top and bottom of each diagonal once on the outside and once on the inside of the standard; thus he created a kind of structural unit, which the engineer Robert Milbourne (dates unknown) described as a

> *"(…) cylindrical cantilever (…)"* (Milbourne, 1923, p. 2).

This revolutionary braced structure was built in 1880 in the Old Kent Road, London for a three-lift gasholder (v= 155,743m3) and consisted of 28 I-shaped, riveted standards (h= 48,77m), four rings of intermediate and one top girder, and diagonals of flat iron strip. Given his contribution and importance to the building of gasholders, this type was called **Livesey pattern gasholder.**
Only a few years later, again the Livesey brothers, designed a further development of the previous guide frame by superimposing the single and the double cross into the **three cross bracing [fig. 4-D]** guide frame that had a filigree, star-like appearance. This concept was only realised once, in 1889, in East Greenwich, London for the very first four-lift gasholder (v= 232,198m3), which had a six-tiered guide frame **[fig. 7]**. This eye-catching structure was composed of 28 riveted wrought-iron, I-section standards (h= 60,35m) – of which the inner flange was much larger than the outer[17] (Tucker, 2014).
Both Livesey-types were fitted with a horizontal wind brace in the top ring. These were first introduced by John Birch Paddon (1825-1910) in 1876, when he attempted to stiffen the three-tiered guide frame (h= ca. 22,20m) of a gasholder built at Hove against the strong wind conditions there. This

[16] Frederick Southwell Cripps (1855-1941) was an important figure in the introduction of calculations; see his report "The Guide Framing of Gasholders", 1899.

[17] Demolished in 2020.

construction element was later used in many braced structures and known as **Paddon's wind ties** (Newbigging, Fewtrell, 1879).
The engineer Samuel Cutler Jnr. (1869-1937) optimised Livesey's development in 1888 with a patent for a new **triangulated bracing system [fig. 4-G]**, which no longer required horizontal girders. Standards and diagonals were formed of I-shaped trussed girders. This thus allowed a very efficient construction and a serial production of the Cutler-type, that was distributed all over the world with an uniform appearance, that became a trademark for Cutler and his company. In 1889, the first Cutler-type guide frame (h= 21,50m) was built in Tunbridge Wells for a two-lift gasholder **[fig. 8]** (Cutler, 1888).
In parallel with the triangulated type, another design enriched the variety of guide frames. The **detached frame [fig. 4-E]** also had no horizontal girders as well and united all tiers of single cross positioned bracings to the standards forming a large, vertical bay. It was Cutler, who first mentioned this design in his patent of 1888. However, the best known and largest gasholder of Great Britain had a detached frame (h= 54,86m) was built in 1892 by the Livesey brothers in East Greenwich, London **[fig. 6-E]**. The guide frame was composed of 28 riveted wrought-iron, I-section standards with five single crosses made with T-section diagonals in each bay (Tucker, 2014).
The braced structure in particular had many special forms, depending on the engineers of the different countries, e.g. the design by the company August Klönne[18], that repositioned the guide rail of a single cross braced structure from the axis to the middle of the standards **[fig. 4-B]**. An early Klönne-type guide frame was realised in 1903 in Wernigerode for a single-lift gasholder (v= 2,500m3), which was converted to a telescopic gasholder about 20 years later by the addition of a second lift and tier (Anonymus, 1929). This guide frame was widely used in Germany and abroad and was easily recognisable due to the unique and prominent position of the guide rail.

[18] The company MAN started to build hydraulic gasholders with their own frame design in 1874. An elegant filigree design was realised in Vienna in 1908/09: a four-lift gasholder (v= 150,000m3) with a diamond-shaped frame (h= 66,00m, incl. a drop-shaped tank) (MAN, 1909a).

3. *Water tank*

The water tank ensured the gas-tightness of the gasholder. Its position varied from overground to being fully or partially underground, depending on the conditions of the ground. Some designs also included banks surrounding the tank wall. The straight cylinder was the most common tank form built. However, the hidden inner shape and the bottom underwent their own development.

a) Flat bottom tank

The first standard tank forms completely retained the geometry of a cylinder[19] – including a flat bottom **[fig. 5]**. Depending on conditions of the soil – such as the London clay with its advantageous sealing properties – it was quite common for the tank to be constructed partially or totally immersed in the ground. The tank consisted of a circular wall built of one, and later two layers of masonry or in combination with concrete – the latter, introduced in 1871 by George Livesey[20] – and was usually covered with puddle on the outside. The columns or standards were fixed on external pilasters, that were added on the outside of the circular wall; larger diameters even had secondary pilasters between the primary ones. The bottom of the tank was composed by layers of clay, bricks or cement.
This building technique proved itself over a long period for both, smaller and larger tanks: Clegg Jnr. showed in his treatise in 1841 an example with a diameter of 27,67m (h= 7,62m). Charles Hunt (1842-1929), on the other hand, planned a tank with double the capacity and a diameter of 61,00m (h= 11,00m). These large tanks were considerably strengthened by the introduction of cement mortar and wrought iron tie rods (Newbigging, Fewtrell, 1879).
Overground or slightly immersed tanks were used less frequently, mainly for smaller and medium-sized iron tanks, and recommended by Samuel Hughes (dates unknown) for diameters up to 30,00m (h= 8,00m) (Hughes,

[19] In 1810 B. Cooks even introduced wooden barrels – tightened with wrought iron tie rods – for smaller tanks (Copper, 1816). The design was limited in size. One of the last barrels was demolished in 1843 at the gasworks in Brick Lane, London (Newbigging, Fewtrell, 1879).

[20] This composite tank (d= 46,43m; h= 11,58m) was built in Old Kent Road, London.

1853). However, this position was also preferred in locations with complicated ground conditions or in areas with high groundwater levels. The tanks were composed of:

- Cast-iron plates and wrought-iron tie rods – used until the mid of the 19th century. An example of this design still exists in Fakenham with a single-lift gasholder built in 1888.
- Wrought-iron plates – which replaced the cast iron predecessor and provided a much lighter and more efficient structure. A large example was demolished 2017 in Poplar, London: a four-lift gasholder (v= 104,772m3) built in 1928 with an overground riveted wrought-iron tank (d= 58,50m; h= 11,40m). (Tucker, 2014)
- Masonry – was rather rarely used for the overground position[21].

For the iron constructions bearing brackets or outer standards were attached to support the guide frame. The disadvantage with this type was the difficulty of riveting the sheets of greater thickness, that were required for larger tank sizes. In 1908 MAN recommended riveted iron tanks up to a maximum of 100,000m3, because 150,000m3 would require lower mantle sheets of 40mm thickness (MAN, 1908).
All tanks were equipped with low resting posts or stones for the empty state of the lifts. Depending on the construction of the crown, the tank was fitted with a kingpost and/ or a rest frame[22].

b) Frustum-shaped tank

Larger dimensions of underground tanks led to massive ground excavation. The frustum-shaped design **[fig. 5]** reduced not only the excavation works, but also the water capacity and king posts or frame structures if required. In general, the outer wall and the ceiling consisted of the same construction methods as the *flat bottom tank*, but these were mostly used for totally or partially underground tanks.

[21] Another monolith building technique was introduced with reinforced concrete at the beginning of the 20th century (see Rehm, 2019), and was also occasionally used for tanks in an overground position.

[22] The flat bottom tank was also used in an elevated position for water tower s. The tank was first executed in iron and, at the beginning of the 20th century in reinforced concrete (see Berger, on-going).

The height and the ratio of the frustum – also called the *dumpling* – itself were chosen according to local ground conditions. Usually they were built to a height, that was at least half or the same as the tank, with a maximum ratio of 1:1. Lower frustums were used more rarely.
In 1841, Clegg Jnr. proposed this design for tanks with a diameter of 18,00m and more and described an early single-lift gasholder, whose frustum reached the same height as the tank, namely 7,62m (d= 26,67m). The frustum's base extended almost across the entire bottom and the sides were terraced in the middle.
It was George Livesey again who, in 1875[23], used a water tank built entirely of concrete at his gasworks in Old Kent Road, London. This was a further development of the composite building technique he had used previously. This tank (d= 56,10m; h= 14,33m) also stood out for its extraordinary form: the frustum was combined with a second circular wall along the upper edge (d_{outer}= 28,66m). This resulted in a ring-shaped form, that reduced the water content even further. A third circular wall (d_{inner}= 3,05m) served as a king post for the crown. After the first winter periods, Livesey discovered cracks, which prompted him to reinforce the mass-concrete wall with wrought-iron tie rods.
The introduction of concrete tanks was one of the pioneering milestones in gasholder construction (Clegg, 1841; Newbigging, Fewtrell, 1879).

c) Spherical bottom tank

In general, the spherical bottom tank **[fig. 5]** had many similarities to the frustum-shaped design in terms of the construction methods and positions relative ground level, as well as the advantages regarding less excavation and amount of water. However, the geometry of a hemisphere or even a segment was much more elaborate to model than a frustum. The rise of the spherical bottom varied and sometimes reached half the height of the tank.
In 1860, a fairly flat spherical-bottom tank (d= 49,76m; h= 10,67m) was planned for a gasholder in Kensal Green, London. It was built fully underground with a circular wall of masonry; in the centre was a low king post for the crown (Western Gas Light Company, 1860).

[23] J. Douglas form Portsea presented a paper in 1874 on the use of concrete for gasholder tanks (Newbigging, Fewtrell, 1879).

Overall, the spherical-bottom tank was rarely used, as the efficient frustum design prevailed.

d) Supporting bottom tank – Intze-type

A very ingenious form derived from the spherical bottom design, was invented by the German hydraulic engineer and Professor Otto Intze (1843-1904). This was the supporting bottom tank, that he had first developed for water towers and then adapted for gasholders; both were patented in 1883 (Intze, 1883a; Intze, 1883b).
The characteristic pointed supporting bottom tank was created by the intersection of a sphere with a cone. This tank, referred to as **Intze-type I,** was conceived as an overground tank[24] that needed only a ring-shaped foundation. This led to new possibilities in gasholder construction, a new form for the underside of the tank, and also the creation of additional space, for example, for storage.
As early as 1884, Intze optimised the supporting bottom basin for tanks with larger diameters, as their spherical bottoms also required a larger radius. This prompted Intze to lower the apex, the **Intze-type II**[25]**.**
A remarkable gasholder (v= 37,000m3) with an Intze-type tank was built in 1889 at the steelworks of Krupp in Essen a. d. Ruhr: the supporting bottom tank was conceived as a complex multiple folded structure with inner columns and a central crown (Anonymus, 1900).
The Intze-type remained mainly a German phenomenon and was built mostly in German-speaking countries.

e) Annular tank

The annular tank **[fig. 5]** reduced the amount of water to a minimum. By inserting a second circular wall, the basin was reduced to an annular channel. The width was determined by the number of lifts and space they occupied when the gasholder was empty.

[24] Intze first used a cone before he adopted the spherical shape; the first Intze-type I was built 1884 in the water tower in Remscheid (v= 400m3) (Anonymus, 1900).

[25] Both, the Intze type I and II were also used for water towers. Because of the overhanging tank design, the tower shaft was noticeably recessed (see Berger, on-going).

The position of the annular tank was totally or partially underground [26], especially in difficult ground conditions, e.g. rock (Milbourne, 1923). In general all construction methods – masonry, concrete and iron – suited to the annular type, although the prevailing material was iron.
Nevertheless, the initial idea of the annular tank was even used in 1818 by Stratton in his doubled gasholder, but ultimately it was much more elaborative and expensive than for example the frustum design.
An extraordinary example was built in 1902, for a gasholder (v= 100,000m3) in Amsterdam, that was, for many years, the largest of its kind. Built next to three existing underground tanks, a new overground tank (d= ca. 63,00m) was required. For this, August Klönne chose the annular form with an additional base of masonry (Verhagen, 2014; Anonymus, 1929).

f) Drop-shaped tank

Jagschitz from MAN contributed a very efficient shape and solution for flat bottom tanks made of wrought iron. He solved the difficult edge connection between the mantle and bottom sheets by creating a completely new geometry: the drop-shaped tank **[fig. 5]**, which he patented in 1909. A continuous transition from the shape of a drop of liquid to the horizontal, was created by the prevailing hydrostatic pressure itself. Thus, the wrought-iron plates had a single, constant thickness. The main construction challenge was at the place where the iron tank was supported on the foundations. The shell was supported by curved lattice standards (MAN, 1907; MAN, 1909b).
An outstanding surviving pair of drop-shaped tanks can be seen in Augsburg-Oberhausen. They were built between 1910 and 1915: the first with a diameter of 39,80m and the second, 53,40m (MAN, 1909c). The time between the date of patent and the completion of the first example was particularly short.
MAN recommended this design for gasholders with a capacity up to 1,000,000m3. Drop-shaped tanks were mainly built in Germany (MAN, 1920a).

[26] One exception was the MAN's overground annular tank, that had even an elevated position over a kind of basement level (MAN, 1909a).

4. *Enclosing structure (gasholder house)*

Especially in areas with severe winters it was very important to prevent the sealing water from freezing. This requirement led to the erection of massive enclosing structures that allowed heating of the water. These structures came to have another function, to disguise the industrial character of the gasholder in a city environment such as Berlin. Thus, the gasholder itself was no longer visible.
The second shell usually had a cylindrical ground plan, the circular of which was made of masonry and/or reinforced concrete[27]. This wall allowed the structure of the guide frame to be reduced and also provided access walkways on the inside. Due to their often ornate and articulated façade (windows, cornice, pilaster, etc.) these enclosing structures resembled sacred buildings rather than functional buildings. The roof was conical or pyramidal or a crown and was built of iron with a lantern at the top for ventilation.
An exceptional example by Schwedler is represented by the gasholder (v= 30,000m3) in Fichtestraße, Berlin[28], which was built in 1876 with an enclosing structure (d_{inner}= 54,60m; h_{wall}= 21,99m). Its crown was designed with Schwedler's revolutionary lattice structure (Anonymus, 1876)
Although associated with the mid- to late 19th century, in fact early treatises showed, that enclosures were used in some very early gasholders. From 1817 onwards, gasholders were mostly built free-standing without enclosing structures (Accum, 1819; Clegg, 1841).

5. *Piston-type gasholder*

Due to the new sealing technique used in piston-type gasholders, they had a completely different shape and construction elements.

[27] Polygonal shells were also built with iron skeleton constructions and filled with masonry; e.g. the enclosing structure in Nuremberg, that was erected in 1891 for a gasholder (v= 18,500m3). The roof was a Schwedler-type crown (Terhaerst, 1906).

[28] This gasholder is now called "Fichtebunker" and was the first to be reused: During the second world war it was con-verted into a bunker. Today it houses a museum and penthouses have been built under the crown (see Berliner Unterwelten, 2022).

a) MAN-type – polygonal ground plan

The prototype by Jagschitz (see chapter III.3.) had a polygonal ground plan and was built in 1915 in Oberhausen, Augsburg with a capacity of 1,600m3. It consisted of 12 I-section standards (h= 15,25m) with straight flanged plates in between (l= 2,96m) and a piston with radial trussed girders. This new lightweight structure was protected by an enclosing structure of reinforced concrete[29] (MAN, 1920b).
The standard design was freed from the secondary shell and had a characteristic and uniform appearance. The gasholder was accessible via an external staircase and/or elevator leading to the top, where a foldable ladder or an elevator made it possible to enter the piston. The shell was stiffened by the foundation ring and the pyramide-shaped roof made with radial trussed girders. According to the necessary capacity, the shell and the number of its standards, the piston and the roof were adapted to the required dimensions. Thus the MAN-type represented a very efficient structure due to the serial prefabrication and the quick construction process on site (MAN, 1930; Leffer, 2006).
The largest MAN-type gasholder was built in Chicago in 1928 **[fig. 6-F]** with a capacity of 566,000m3 and a height of 114,00m, and completed in only five months. Unfortunately, this gasholder was destroyed by plane crush in 1943 (MAN, 1930; MAN, 1956).
Today, the Hans Leffer company owns the patent and still builds and repairs both MAN-type and hydraulic gasholders used in steelworks.

b) Klönne-type – circular ground plan

The different and competing sealing technique of the Klönne-type (see chapter III.3.) led also to a different structure and shape. The gasholder patented in 1926 based on a circular ground plan and the curved plates were attached to the inner flanges of the I-section standards. The Klönne-type used flat crowns for both the roof – as a trussed structure – and the piston. The latter was designed as a suspended flat crown with supporting radial trussed girders along the edge that were designed in a significantly reduced

[29] See Rehm, 2019. Today the gasworks in Augsburg still represent an impressive ensemble with their enclosing structure of the prototype, a MAN standard-type and two hydraulic gasholders. The site is now to be converted and reused!

way compared to the MAN-type. To support the piston when empty, the bottom was also spherical (Ansaldo, 1936).
The largest Klönne-type and dry-sealed gasholder was built in 1929 in Gelsen-kirchen for the Nordstern colliery. With a capacity of 600,000m3 this gasholder had a majestic height of 149,00m. Unfortunately, it was demolished during World War II (Gross, 1960).

In contrast to the hydraulic gasholders, which had a hundred years of history and development behind them, as well as a wide range of different designs, the dry-sealed gasholders characterised the cityscapes and landscapes with their uniform appearance, especially the MAN-type **[fig. 6]**.

V. *Conclusion*

Gasholder construction in the 19th and early 20th centuries shows a multi-disciplinary history: the introduction of the revolutionary gas lighting resulted in an increasing demand for gas. The shape of the gasholder is driven primarily by its *function*: the water-sealing technique led first to a complex composition of different construction elements, which each underwent its own development and refinement reflecting progress in *iron technology*, combined with the imaginative empirical work of *gas engineers*. These engineers effected major improvements through the use of theoretical calculations in order to design more efficient and larger structures. The hydraulic gasholders reached the pinnacle of its development at the end of the 19th century by which time a wide range of different forms were in use.
The dry-sealed gasholder marked a new era in gasholder construction from the early 20th century because of a very reliable technique with an uniform, serially prefabricated structure, that varied just in size according to their capacity.
Finally this building type represents an important testimonial of industrial heritage and structural engineering, that demonstrates the potential of iron construction in this period, much as the development of iron bridges during the same time. Furthermore the technical and structural design process did not stop, but continued to develop further as an essential storage vessel for various purposes, even in the 21st century.

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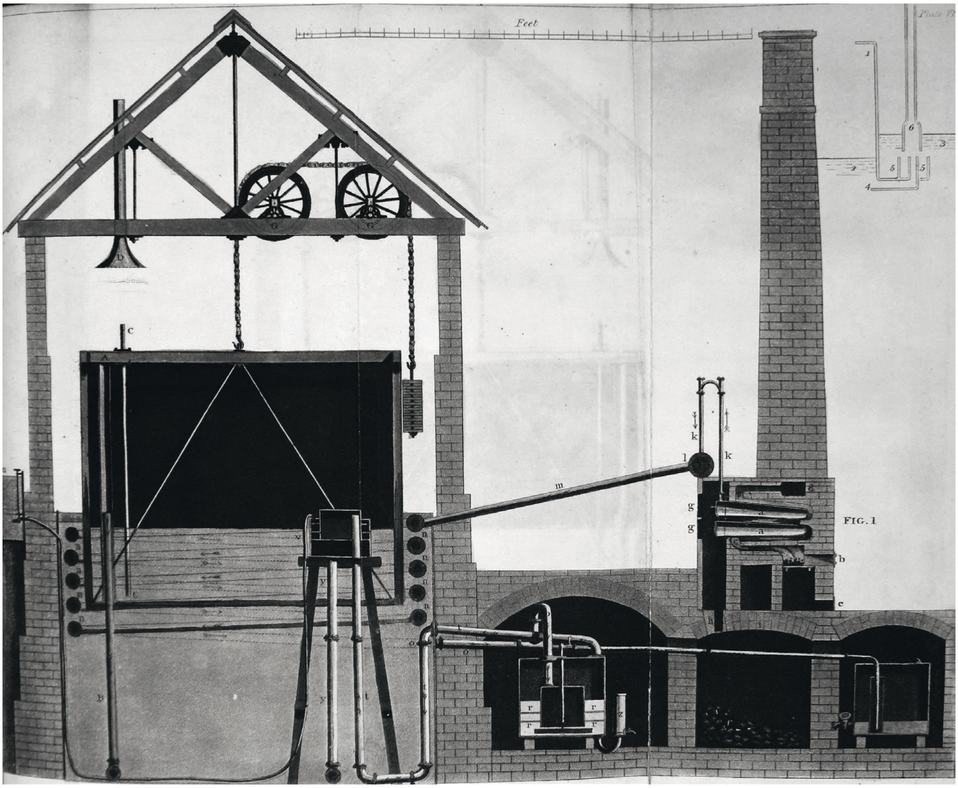

Fig. 1. Clegg's gasworks design for cities (Accum, 1816).

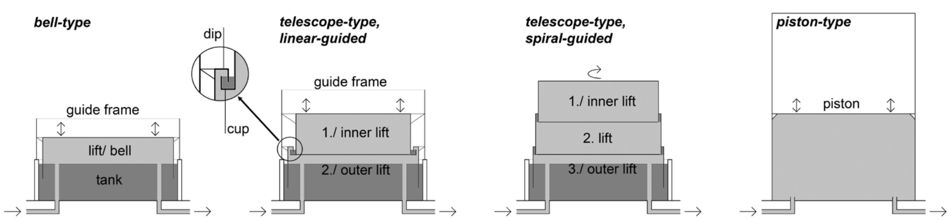

Fig. 2. Development of gasholders – sealing technique (Berger, 2018).

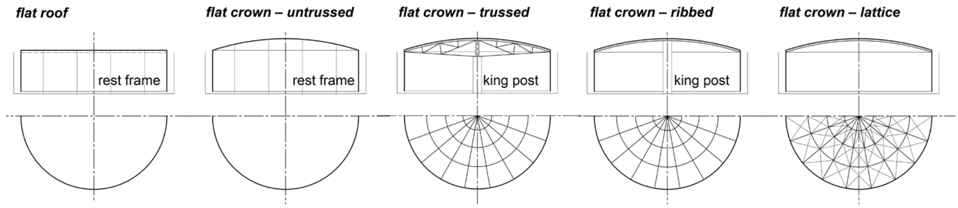

Fig. 3. Flat roofs and flat crowns (Berger, 2019).

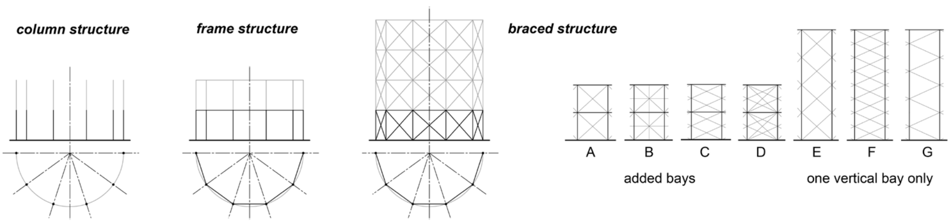

Fig. 4. Guide frames (Berger, 2019).

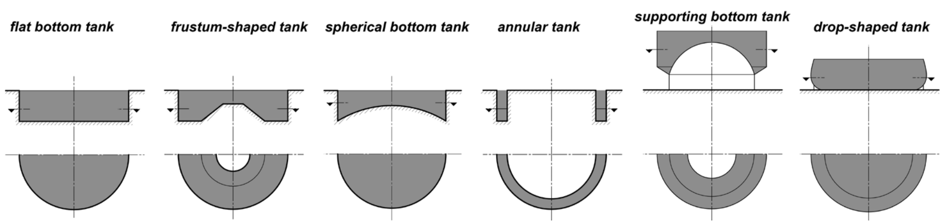

Fig. 5. Water tanks (Berger, 2019).

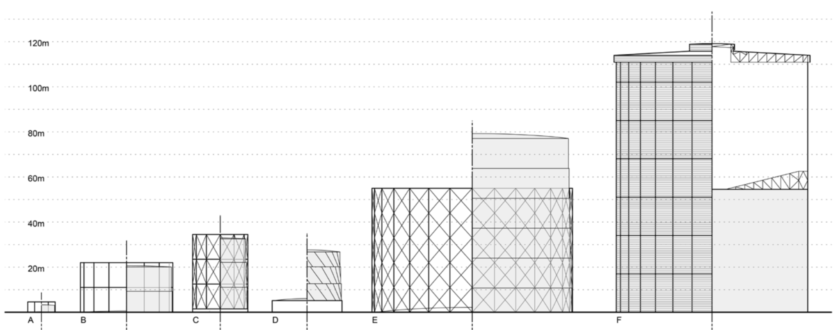

Fig. 6. Construction history of gasholders – overview (Berger, 2019).

Fig. 7. Gasholder in East Greenwich, London (Berger, 2015).

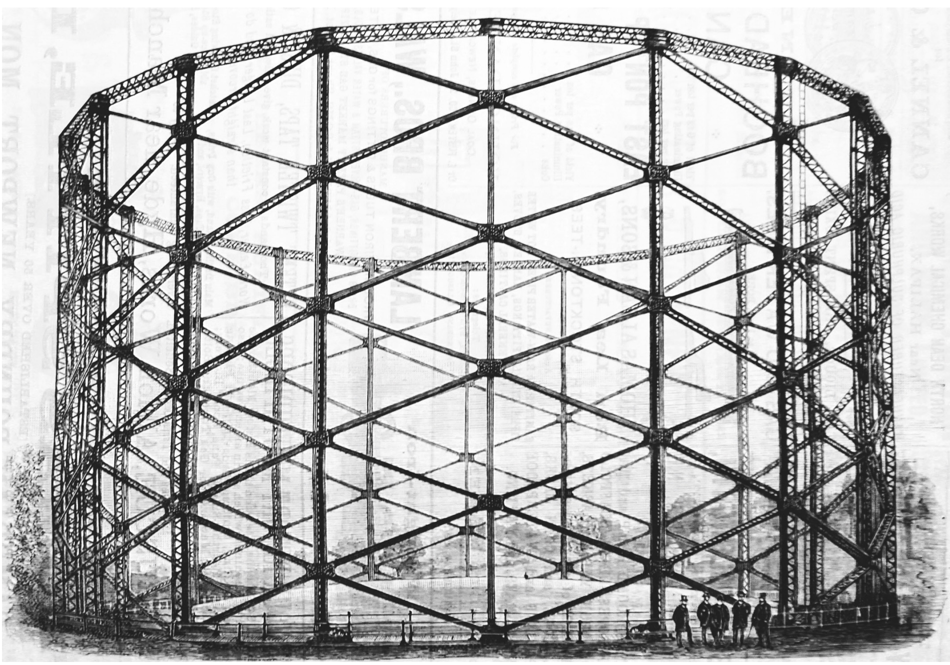

Fig. 8. Gasholder in Tunbridge Wells (Cutler, 1890).

Gianluca Capurso

Struttura e architettura. Strumenti e risultati di una ricerca sul dopoguerra italiano

Nell'Italia del dopoguerra, il rapporto tra ossatura portante ed espressione architettonica assume un ruolo centrale nella progettazione, stimolando fruttuose collaborazioni tra architetti e ingegneri. Al modo in cui questo nesso si concretizza in alcune opere, significative e già famose nella storia dell'architettura italiana, è dedicato il lavoro di ricerca sul tema "Struttura e architettura. Indagini sul dopoguerra italiano", che ha ricevuto la special mention dell'edizione 2019 del Premio Edoardo Benvenuto[1].
La ricerca è stata condotta nella più ampia cornice di studi sulla storia della costruzione italiana del Novecento svolti a cura del gruppo di ricerca "Architettura e costruzione", coordinato da Sergio Poretti e da Tullia Iori all'Università di Roma Tor Vergata.
Nel lavoro sono prese in esame, in particolare: la palazzina "il Girasole a Roma" di Luigi Moretti (1947-1951), la torre Velasca a Milano dei BBPR e Arturo Danusso (1950-1957), la villa "La Saracena" a Santa Marinella di Moretti (1955-1957), il grattacielo Pirelli a Milano di Gio Ponti, Pier Luigi Nervi e Danusso (1954-1960), il palazzo della Regione a Trento di Adalberto Libera e Sergio Musmeci (1953-1966) e la Stock Exchange Tower a Montreal di Moretti e Nervi (1960-1965). Opere molto diverse tra loro sia per i temi affrontati – la palazzina, la piccola villa al mare, l'edificio alto, il palazzo istituzionale – sia per gli approcci progettuali dei loro autori, che dimostrano quanto diffuso e trasversale sia, tra gli architetti, l'amore per la struttura e l'interesse per le sue potenzialità in chiave linguistica, figurativa o compositiva.

La ricerca si è basata sulla bibliografia disponibile, consistente data la notorietà delle opere e, soprattutto, sulla documentazione originale relativa alla progettazione, agli appalti e alla costruzione, oltre che sulle informazioni relative agli interventi condotti sugli edifici dopo il loro completamento.

[1] Il lavoro "Struttura e architettura. Indagini sul dopoguerra italiano" è stato poi pubblicato, con lo stesso titolo, per l'editore Gangemi (Roma) nel 2020.

Le attività condotte per reperire questi materiali sono state particolarmente entusiasmanti. Alcune sono state svolte in archivi istituzionali, pubblici, che conservano fondi di progettisti già più o meno esplorati dagli studiosi di storia dell'architettura, che stavolta sono stati sottoposti alla lente d'ingrandimento della storia delle costruzioni, alla ricerca specifica di informazioni sulle tecnologie esecutive e le soluzioni di cantiere. Per il peculiare carattere della ricerca, è stato necessario, però, cercare informazioni anche negli archivi dei comuni e delle Prefetture, che conservano pratiche edilizie e documenti di collaudo delle strutture di cemento armato; dei fotografi, ingaggiati per immortalare le opere e la loro costruzione; delle imprese, seppure non sempre facilmente consultabili; presso famiglie ed eredi di progettisti o costruttori, che si sono resi disponibili a riscoprire documenti conservati in qualche baule nelle abitazioni private. Anche gli archivi on line hanno riservato molte sorprese, grazie alle notevoli potenzialità degli strumenti di ricerca offerti dalla rete.
Dal punto di vista operativo, è stato fondamentale l'uso del "ridisegno ricostruttivo", uno speciale spaccato assonometrico, progettato appositamente per focalizzare il tema di interesse nello studio dell'opera, il cui impiego nelle ricerche sulla storia della costruzione italiana del Novecento è stato messo a punto da Poretti già negli anni ottanta.
Per ciascuna delle opere analizzate sono stati elaborati ridisegni dedicati, sia di insieme sia di dettaglio, per mezzo dei quali è stato indagato il rapporto instaurato tra ossatura portante e componente architettonica. Non si tratta di semplici rappresentazioni a posteriori di quanto appreso, nonostante l'efficacia comunicativa delle elaborazioni grafiche sia stata costantemente curata e progressivamente testata in occasione di presentazioni pubbliche sul tema della ricerca in convegni nazionali e internazionali. Come ormai consolidato nell'esperienza del gruppo di ricerca, i disegni sono stati usati, piuttosto, come veri e propri strumenti di studio, un'attrezzatura di laboratorio per elaborare le conoscenze attinte dalle fonti, al fine di ricostruire le caratteristiche originarie delle opere e metterne a fuoco alcuni tratti peculiari.
Proprio per sfruttare al massimo le potenzialità analitiche dello strumento, ogni ridisegno è stato quindi progettato, una volta acquisite le informazioni utili per la comprensione della singola costruzione, continuamente revisionato e migliorato parallelamente all'avanzare degli studi. Nel corso del lavoro è emersa anche l'idea di riportare i risultati della ricerca nella forma di legende analitiche, descrittive delle caratteristiche tecnologiche di ciascuna parte della costruzione, nonché di accompagnarle con ulteriori

disegni, di maggiore dettaglio, in cui la singola componente analizzata nel testo è evidenziata tramite un espediente grafico.

Gli studi condotti hanno offerto anche spunti per altre ricerche, condotte negli stessi anni dall'autore sulla storia della costruzione e dell'ingegneria italiana del Novecento. Dalle analisi svolte sui due grattacieli milanesi è stato sviluppato infatti lo studio sul "grattacielo all'italiana", che ha indagato il particolare modo in cui l'ossatura portante è utilizzata dai progettisti, negli anni cinquanta e sessanta, per caratterizzare, rispetto al modello internazionale coevo, il linguaggio e l'immagine degli edifici alti (Capurso, 2020). Inoltre, dalla ricerca sulla Stock Exchange Tower sono stati ampliati, nell'ambito del progetto SIXXI sulla Storia dell'Ingegneria Strutturale in Italia del XX Secolo, in corso presso l'Università di Roma Tor Vergata[2], gli studi sull'ingegneria italiana all'estero nella seconda metà del Novecento (Capurso, Martire, 2020).

Due grattacieli inconfondibili

In seguito a una prima ricognizione bibliografica, due edifici sembravano costituire un terreno fertile per iniziare gli studi: il grattacielo Pirelli e la torre Velasca, i più famosi grattacieli italiani del Novecento, sempre citati nei volumi di storia dell'architettura.
Per analizzare il Pirelli[3] era necessario recuperare le pubblicazioni disponi-

[2] Il progetto SIXXI, coordinato da Sergio Poretti e Tullia Iori, è stato finanziato con un ERC Advanced Grant (<www.sixxi.eu>).

[3] La società Pirelli avvia lo studio per realizzare la sua nuova sede, di fronte alla stazione centrale di Milano, già nel 1952. L'anno successivo incarica Gio Ponti, insieme ai suoi associati Antonio Fornaroli e Alberto Rosselli e allo studio guidato da Egidio dell'Orto e Giuseppe Valtolina. Nell'ottobre del 1954 è pronto il progetto di massima, che già prevede una torre più alta di 120 metri. In seguito al coinvolgimento di Nervi e Danusso, la struttura è completamente riprogettata e alla fine del 1955 sono testati con successo all'Ismes sia una campata di solaio, riprodotta in scala 1:5, sia l'intero manufatto su un modello 1:15, alto dieci metri. All'inizio del 1956 Nervi passa a verificare analiticamente e geometricamente le membrature, anche alla luce dei risultati delle prove e, mentre si definiscono gli ultimi dettagli esecutivi, i lavori prendono il via. La struttura, eseguita dalla ditta Bonomi & Comolli Silce, è terminata nell'estate del 1957 e collaudata l'anno successivo. Il curtain wall è progettato con l'attenta cura di Rosselli e la consulenza tecnica delle ditte Curtisa di Bologna e Fratelli Greppi di Milano. Anche per la presenza di numerosi pezzi speciali, il completamento dei prospetti richiede più di un anno di lavoro, durante il quale

bili, soprattutto quelle relative agli anni della realizzazione, ma anche condurre ricerche in alcuni archivi, che si sono rivelati uno più bello dell'altro: in quello della società Pirelli, intanto, che custodisce foto della costruzione dell'edificio e immagini dell'opera completata – spesso adornate dalle pubblicità dell'azienda committente, così evocative di quegli anni – nonché documenti di cantiere utilissimi per la ricerca; nell'archivio dell'Ismes - Istituto Sperimentale Modelli e Strutture di Bergamo, consultato, ancora in fase di riordino e catalogazione, con il prezioso aiuto del personale che vi lavora; nei fondi di Ponti e di Nervi, ovviamente, conservati presso il Centro Studi Archivio della Comunicazione di Parma e al MAXXI di Roma; nell'Archivio di Stato di Milano, dove è confluita la sezione "Cementi armati" dell'archivio della locale Prefettura; nell'Archivio Comunale - Ripartizione edilizia privata, in cui, insperabilmente, sono conservati disegni, non rinvenuti altrove, relativi sia alle soluzioni costruttive della facciata, sia a quelle esecutive strutturali. Si sono aggiunti poi: le foto messe a disposizione con generosità dalla famiglia Comolli, erede dell'ingegnere titolare dell'impresa costruttrice dell'edificio, e il materiale conservato presso il fondo cinematografia scientifica del Politecnico di Milano, relativo alle pionieristiche prove in galleria del vento condotte sul grattacielo. È stato così possibile ricostruire non solo l'evoluzione del progetto architettonico, ma anche di quello strutturale e delle soluzioni costruttive ipotizzate e poi messe in atto.

A valle delle esplorazioni condotte, e del lavoro al tavolo anatomico del ridisegno, è emerso che nel Pirelli il ruolo degli ingegneri non si limita al calcolo strutturale: Nervi, in particolare, inventa l'assetto che permette di realizzare l'edificio snello ed elegante desiderato dall'architetto milanese, alto 126 metri, ma profondo appena 18. Il problema statico principale derivante da queste proporzioni è, evidentemente, la resistenza del fabbricato al vento. Per affrontarlo Nervi, invece di utilizzare una gabbia ordinaria di pilastri e travi, sceglie di far funzionare la torre come una struttura muraria, la cui stabilità è garantita limitando l'eccentricità della risultante di carichi e spinte sulle sezioni orizzontali resistenti. Per questo motivo concentra le strutture verticali in pochi, grandi setti, mentre le grandi campate che derivano dalla nuova impostazione del progetto sono coperte con

si realizzano anche impianti e finiture interne. La società Pirelli trasferisce i suoi uffici nella nuova sede, ormai perfettamente funzionante, nel marzo del 1960.

solai dotati di nervature, alte 75 centimetri, a sezione variabile. L'originale sistema strutturale, che Nervi definisce "a gravità" (Nervi, 1960), è sfruttato da Ponti per rendere inconfondibile l'immagine del grattacielo.
L'architetto, affascinato tra l'altro dalle sperimentazioni su modello condotte presso l'Ismes – dove se ne testa uno, spettacolare, alto quasi 10 metri, che riproduce l'intera torre (Oberti, 1955) – fa comparire infatti nei prospetti le coppie di setti rastremati con l'altezza, perfettamente funzionali a sottolineare la "forma finita" dell'edificio, da lui sempre ricercata (Ponti, 1956). Per raggiungere lo scopo, persino il curtain wall, di produzione industriale, è minuziosamente adattato al profilo dei pilastri, grazie all'introduzione di elementi speciali, sempre diversi ad ogni piano.

Mentre si andava completando la ricerca sul Pirelli, è stato avviato lo studio sull'altra torre milanese, la Velasca. Nell'impossibilità di accedere all'archivio dello studio dei BBPR, il lavoro si è basato sulla documentazione grafica già pubblicata (Fiori, Prizzon, 1982) e, soprattutto, sul fondo della SGI - Società Generale Immobiliare, costruttrice della torre, conservato presso l'Archivio centrale di Stato di Roma: nel suo materiale fotografico si trovano album di immagini scattate in momenti decisivi del cantiere, che indugiano spesso sulla realizzazione dell'ossatura. Nel corso della ricerca sono stati esplorati di nuovo anche l'Archivio del comune di Milano e, soprattutto, l'archivio dell'Ismes, in cui sono conservate le relazioni delle prove condotte sui modelli degli elementi strutturali del nucleo, di un pilastro e di un solaio tipo, di fondamentale importanza per la progettazione del grattacielo[4].

[4] L'edificio sorge a pochi isolati dal Duomo di Milano, in un'area colpita da bombardamenti durante la Seconda Guerra Mondiale, nel 1943. Nel 1948 la RICE (Ricostruzione Comparti Edilizi) – una ditta controllata dalla SGI – avvia il processo di acquisizione dei lotti, concludendo l'operazione dopo circa tre anni, mentre già si approntano i progetti dei nuovi fabbricati, con il coordinamento del gruppo di architetti BBPR. Per valorizzare l'intervento, la RICE contratta con il Comune la realizzazione di un edificio alto, non previsto dal Piano regolatore, affiancato da due blocchi di nove piani, in cambio del quale l'amministrazione ottiene una riduzione della volumetria edificabile e la cessione gratuita di aree per viabilità e parcheggio. Il ritardo nell'approvazione al piano di ricostruzione dell'area posticipa l'inizio dei lavori di fondazione fino al 1954, mentre per preoccupazioni di ordine economico della società committente i getti al di sopra della quota stradale sono avviati solo all'inizio del 1956. Al ritmo record di un piano ogni dieci giorni circa, la struttura è completata e nel febbraio 1957 si possono iniziare le operazioni di finitura, ultimate nell'autunno dello stesso anno.

Nel frattempo, l'elaborazione del ridisegno rivelava alcuni caratteri distintivi della torre. Con una sezione verticale lunga tutta la facciata e facendo "sparire" una parte della batteria di solai è possibile infatti visualizzare l'originale soluzione statica che consente di realizzare la parte superiore dell'edificio, espansa rispetto allo stelo che la sostiene, e lo speciale rapporto instaurato tra la struttura e le componenti del prospetto.

Quando i BBPR sono incaricati dalla SGI di progettare la torre si orientano, inizialmente, verso l'uso di uno scheletro metallico, apparentemente più ovvio per un edificio di 25 piani. Da un'accurata analisi tecnico-economica commissionata a uno studio di consulenti nordamericani risulta, però, che l'ossatura metallica costerebbe il 25% in più rispetto a quella in cemento armato (Golinelli, 1958). Pesa negativamente sull'opzione della costruzione in acciaio anche la persistente arretratezza tecnologica del cantiere nazionale nel dopoguerra, nonché la complessiva inesperienza dei nostri ingegneri (Pifferi, 1959).
Si opta, quindi, per uno scheletro in calcestruzzo, che viene agevolmente dimensionato e calcolato dai tecnici dell'impresa costruttrice – la "Sogene - Società Generale per lavori e pubblica utilità", fondata nel 1945 come settore "costruzioni" della SGI – e da Arturo Danusso. È basato su uno schema a pilastri esterni, disposti sulla facciata ogni otto metri circa, collegati da travi perimetrali parapetto e irrigiditi da un nucleo controventante. La forma, a doppia T, della sezione orizzontale di questi setti ne ottimizza il comportamento strutturale a mensola verticale, incastrata a terra. Colpisce, invece, come la planimetria della torre non mostri alcun rispetto per la logica della griglia, facendo affidamento, piuttosto, sulla flessibilità del solaio laterocementizio, in perfetta continuità con il cantiere tradizionale (Capurso, 2020).
La scelta della tecnologia costruttiva usata per la struttura ha importanti conseguenze sull'immagine dell'edificio, più evidenti se volgiamo lo sguardo alla produzione nordamericana contemporanea. Sono gli anni in cui la Lever House di Skidmore, Owings e Merrill, completata nel 1952, influenza gli altri grattacieli newyorkesi in corso di costruzione, diffondendo il linguaggio International Style. Prendendo le distanze dai volumi stereometrici, rivestiti da un curtain wall indifferenziato, in cui il telaio strutturale è ridotto al ruolo di impalcatura permanente ma invisibile a sostegno della facciata metallica, nella torre Velasca i fronti assumono una connotazione muraria e la struttura gioca un ruolo decisivo, ricalcando la caratteristica forma compiuta, "a fungo", della torre. I pilastri, la cui sagoma si assottiglia con l'altezza, secondo un criterio di uniforme resistenza, sono esibiti sulla

parete, alla stregua di costoloni a geometria rigata (ottenuta in cantiere con modelli in gesso al vero e con complicati casseri doppi); i montanti sono altresì raddoppiati in corrispondenza degli spigoli del volume, appositamente smussati, anche se la loro sezione risulta ridotta; le alte travi parapetto, infine, preservano l'immagine piena dell'edificio anche lì dove le tamponature lasciano il posto alle logge delle abitazioni.
A caratterizzare maggiormente la torre è però la soluzione che sostiene l'aggetto del volume superiore che ospita la casa-albergo. Le saette oblique che portano i pilastri perimetrali del blocco espanso, posti in falso rispetto a quelli sottostanti, sono tenute in equilibrio da Danusso grazie a due solai speciali, posizionati al 15° e 18° livello. Se l'inclinazione delle saette introduce, infatti, una componente orizzontale degli sforzi, il primo orizzontamento, realizzato con una piastra laterocementizia doppiamente nervata, funziona come un puntone e il secondo, di cemento armato pieno, in cui è annegata una ragnatela di armature, come un tirante (Cecchi, 1957).
L'innegabile predilezione per la forma finita e per il realismo costruttivo, nonché l'orgoglioso rapporto con la storia e la stretta integrazione con i settori più avanzati dell'ingegneria strutturale (Poretti, 2007), trovano puntuale espressione nella Velasca, che assume il ruolo di una delle opere più emblematiche della costruzione e dell'architettura italiana di questo periodo.

Piccole strutture barocche

Dopo le arditezze strutturali dei due grattacieli milanesi, era il momento di rivolgere l'analisi anche ad edifici che richiedevano, per loro natura, un minore impegno statico, ma dove l'uso dell'ossatura risulta comunque significativo per il tema della ricerca.
Come è modulato dagli architetti il rapporto tra struttura e architettura, per esempio, nella palazzina romana, ovvero in quel particolare tipo edilizio, residenziale, favorito dal piano regolatore della capitale, oggetto nel Novecento di svariate sperimentazioni architettoniche? Tra tutte, l'opera di Luigi Moretti spiccava per originalità e meritava approfondimenti specifici, per la sua posizione distante dal realismo architettonico, dominante nell'Italia del dopoguerra (Rostagni, 2008).
La palazzina "Il Girasole" è stata riconosciuta di fondamentale importanza nell'architettura moderna anche all'estero e ha incuriosito, tra gli altri, architetti come Robert Venturi e Peter Eisenmann, che non a caso l'ha inserita tra i suoi "ten canonical buildings" (Eisenmann, 2008).
Ad interessare, nell'opera di Moretti, era pure la villa "La Saracena", che esibisce sul prospetto principale un gioco di volumi compatti e apparen-

temente impenetrabili, che si dissolvono poi, appena varcato l'accesso, in una sequenza di spazi concatenati, rischiarati da fonti di luce nascoste ricavate tra coperture e pareti, liberamente ritagliate.
Anche in questo caso, dopo una prima ricognizione bibliografica, si è passati a studiare la documentazione d'archivio: il fondo dell'architetto all'Archivio centrale di Stato, che conserva, oltre a relazioni e disegni sui due progetti, anche il materiale iconografico commissionato da Moretti a Vasari, Cartoni e Monti, tra i più importanti fotografi italiani di architettura di quegli anni; i fascicoli relativi alle licenze edilizie delle due opere conservate negli archivi del Comune di Roma e di quello di Santa Marinella; gli studi, gli schizzi e gli appunti relativi alle prime fasi di ideazione di questi capolavori conservati nell'Archivio Moretti-Magnifico. Con un'analisi comparata della documentazione individuata, sono state ricostruite le storie, poco note, dei due progetti e della loro costruzione[5].
Anche a valle delle ricognizioni svolte, restava comunque piuttosto scarna la documentazione disponibile relativa agli aspetti statici, conseguenza del fatto che si tratta di strutture tutto sommato piccole, probabilmente dimensionate secondo criteri empirici. Era perciò necessario compensare la carenza di informazioni desumibili dai progetti con un lavoro di interpretazione, supportato, anche in questo caso, oltre che da sopralluoghi svolti presso i due edifici, dal ridisegno, che si è rivelato uno strumento fondamentale per verificare le ipotesi di studio.

Nel Girasole, Moretti sfrutta tutta la flessibilità del cantiere romano dell'immediato dopoguerra, per sperimentare nuove configurazioni architettoniche.

[5] Nel 1947 il conte Adolfo Fossataro, con cui Luigi Moretti ha da poco fondato la Cofimprese (Compagnia Finanziaria per le Imprese di costruzione e di ricostruzione), affida all'architetto il compito di redigere il progetto della palazzina, da edificare in viale Bruno Buozzi a Roma, nell'elegante quartiere Parioli, in cui ha intenzione di trasferirsi. Moretti prima esplora soluzioni alternative, caratterizzati da differenti gradi di apertura del complesso, poi addiviene alla soluzione "chiusa" della palazzina, sulla base di cui la ditta presenta la richiesta per la licenza di costruzione. Il permesso è rilasciato nel gennaio 1948 e Moretti passa così allo sviluppo esecutivo. In questa fase rielabora alcuni aspetti volumetrici, tra cui il rafforzamento del taglio centrale sulla facciata principale, che diventa molto più profondo del progetto inizialmente presentato, fino al mese di giugno del 1949, quando gli elaborati raggiungono la configurazione architettonica definitiva. La costruzione, dopo una sospensione dovuta alla mancata autorizzazione delle strutture terminali dell'edificio, è completata nel dicembre 1951.

L’ossatura di cemento armato viene quindi adattata senza difficoltà ai layout irregolari degli appartamenti, secondo un criterio diametralmente opposto a quello, divenuto ortodossia del Movimento Moderno, che fa affidamento sulla griglia strutturale modulare, base neutra per la pianta libera (Poretti, 2008). Lo spaccato assonometrico mostra come la struttura non sia concepita come una sequenza di telai, da impostare su luci sempre uguali in una gabbia preimpostata, ma come una serie di solette sovrapposte, sostenute da pilastri e travi le cui sezioni possono variare liberamente a ciascun piano, oltre che da un punto all’altro della planimetria.
Questa versatilità consente a Moretti di utilizzare l’ossatura portante per ottenere specifici effetti plastici, con alcuni accorgimenti che confermano l’amore, sempre dichiarato dall’architetto, per il Barocco. Sul prospetto principale di viale Buozzi le travi emergenti dal solaio e le mensole rastremate laterali sottolineano drammaticamente il peso del volume soprastante, che sembrano sostenere per intero. Quando si tratta di realizzare gli interpiani superiori, le travi sono realizzate invece a spessore di solaio, per non invadere i soggiorni, contro ogni principio di sincerità statica. Sullo stesso fronte, l’effetto di schermo indipendente della facciata è ottenuto disponendo due serie di sottili velette in cemento armato, ancorate ai solai e protese a sbalzo oltre il corpo di fabbrica, su cui si innestano i grandi infissi e il pannello scorrevole in legno che scherma un terrazzino laterale. Lo stesso congegno è impiegato sul retro, mimetizzato stavolta nella tamponatura con l’uso indifferenziato dell’intonaco: ne risulta una parete apparentemente muraria, con le finestre allineate, che però, sorprendentemente, può aggettare dal corpo di fabbrica e inflettersi plasticamente. Infine, all'ultimo piano, i fronti laterali sono ritagliati liberamente da una lunga e stretta asola, lasciata con una estremità libera, grazie all’inserimento di un sottile architrave di cemento armato, sostenuto da un telaio arretrato e posto volutamente in secondo piano.
Con ancora maggiore disinvoltura Moretti impiega le componenti della costruzione nella Saracena[6]: la particolare successione di ambienti che si

[6] Moretti inizia a lavorare al progetto dell’edificio nel marzo 1955, quando prepara una prima planimetria di massima della villa, secondo uno schema “a costruzione abbinata” al lotto adiacente, redatto in base a un accordo tra i proprietari dei terreni interessati, che è approvata dal Comune nell’estate dello stesso anno. Seguono una serie di studi che prevedono diverse varianti al progetto iniziale, rapidamente vagliate da Moretti (Viati Navone, 2012).

inseguono dall'ingresso al mare è coperta con una serie di strutture indipendenti, molte delle quali a sbalzo e con travi estradossate, rese pressoché invisibili all'osservatore (Poretti, 2008).
Nel corpo basso una lunga parete di spina accompagna il visitatore che si dirige verso il soggiorno sul mare. In corrispondenza dell'ingresso, su questa si imposta prima una soletta piena di cemento armato a profilo circolare e sezione variabile rastremata, posta a sbalzo sul giardino. Poi, senza soluzione di continuità, la copertura si raccorda con un solaio impostato sui setti che delimitano l'atrio.
Poco più avanti, dove lo spazio si distende seguendo la finestra aperta sul giardino laterale, la muratura di spina viene distaccata dalla soletta rivelando la presenza di quattro pilastri: come una sottile didascalia, che però non chiarisce del tutto il vero funzionamento del dispositivo statico.
Dall'analisi dei disegni originali si comprende come i montanti siano sormontati da una trave a sezione variabile, su cui si innestano, in direzione ortogonale, mensole rastremate estradossate. Queste, completate con una soletta in cemento armato, coprono tutto l'ambulacro senza che nulla del complesso sistema costruttivo sia decifrabile all'interno. L'equilibrio, visti i quasi cinque metri di sbalzo, è assicurato da due telai, dissimulati nella parete cieca, sul retro dell'edificio.
Giunti in corrispondenza del soggiorno, un nuovo espediente strutturale permette uno straniante effetto di sospensione. L'ambiente sorprende per l'assenza di elementi portanti verticali, che solo a un più attento esame si rivelano abilmente mimetizzati: il muro di tamponamento, il basso recinto curvo della zona pranzo, la luce naturale che filtra nell'ambiente da punti nascosti relegano sullo sfondo la presenza di un setto di quasi 3 metri di lunghezza. Lo sormonta una mensola, estradossata anche in questo caso, che può aggettare così per circa 8 metri e sostenere a sbalzo altre quattro

La struttura dell'edificio realizzato è innalzata su "fondazioni in cemento armato gettato in trincea". Il volume delle camere da letto è sostenuto da una muratura compatta di tufo con ricorsi di mattoni, di spessore variabile, e da solai laterocementizi. La complicata composizione di spazi del corpo basso, invece, richiede una struttura articolata, progettata con il contributo dell'ingegnere Silio Italico Colombini. Nell'estate del 1956 si preparano gli esecutivi per le strutture più impegnative, tra cui la complicata copertura della galleria. Terminati i getti delle strutture, si rende obbligatoria una campagna di rilievi, necessaria per porre in opera gli infissi e eseguire le finiture. Infine, tra febbraio e marzo del 1957 sono definiti gli esecutivi, in scala 1:1, degli infissi della galleria e del salone e nel corso dello stesso anno la costruzione è terminata.

travi secondarie, collegate da solai in cemento armato, che coprono tutto l'ambiente.
Nella Saracena, come nella Casa del Girasole, le possibilità del cantiere italiano del dopoguerra sono messe a frutto per ottenere effetti sorprendenti: dove nessun limite è ancora imposto dalla prefabbricazione industriale, Moretti può rivisitare tutte le consuetudini esecutive, ottenendo raffinate soluzioni spaziali ed effetti scenografici che restituiscono all'osservatore solo frammentarie indicazioni della realtà costruttiva.

Il fascino discreto della struttura

Concluso lo studio sulle due opere di Moretti, la ricerca si è orientata a indagare il modo in cui architetti e ingegneri si dedicano a un altro tema: la grande luce libera. Tra tante sperimentazioni condotte in questi anni, soprattutto negli edifici pubblici, è stata individuata come di particolare interesse una collaborazione tra due progettisti geniali, Libera e Musmeci. L'architetto introduce il tema nella realizzazione di un palazzo istituzionale, la sede della Regione Trentino Alto Adige, senza altri motivi se non quello di condurre una "avventura strutturale" (Musmeci, 1976). D'altra parte, dalla metà degli anni Cinquanta, aumenta tra i progettisti l'interesse per le grandi opere di ingegneria, stimolato dalle realizzazioni degli anni della Ricostruzione, prima, e del boom economico poi (Poretti, 2007).
Per svolgere la ricerca, è stata fondamentale la consultazione dell'archivio dell'ufficio tecnico regionale, ricchissimo anche di documentazione di cantiere, completata da un'esplorazione condotta sull'archivio Musmeci Zanini presso il MAXXI.
Il complesso della Regione è costituito da tre edifici: quello degli assessorati è sollevato su una impressionante serie di pilastri ad albero; il volume del consiglio regionale è risolto con una soluzione a cono rovesciato di cemento armato, che sostiene i posti a sedere; quello su cui si concentrano maggiormente architetto e ingegnere è, però, quello della Giunta, dalla pianta a trapezio allungato, i cui lati di base misurano uno 72 e l'altro 67 metri, che Libera decide di sollevare da terra e appoggiare su due punti, distanti 40 metri l'uno dall'altro[7].

[7] Libera vince un concorso in due fasi bandito dalla Regione Trentino-Alto Adige nel 1953. In seguito al successo avvia una rielaborazione integrale del progetto presentato, pur

In questo caso, con il ridisegno si è voluto analizzare il modo in cui l'originale struttura portante è stata utilizzata nell'immagine, a prima vista piuttosto dimessa, dell'edificio.
Musmeci, coinvolto nella progettazione dall'architetto, abbandona il traliccio centrale a maglie rettangolari inizialmente ipotizzato da Libera per coprire la lunga campata e sostenere l'intera costruzione e lo sostituisce con una lastra piena, che sottopone a uno studio sull'andamento delle linee isostatiche delle sollecitazioni agenti e a un'analisi semplificata del comportamento della parete appoggiata su due punti, così da individuare le aree che possono essere svuotate per realizzare le aperture funzionali (Conzett, 2007).
L'ingegnere imposta poi sulla lastra, ogni otto metri, travi-parete trasversali, a loro volta collegate lungo il perimetro da setti, anch'essi di cemento armato, a costituire un doppio cassone rigido a torsione, che collabora con la parete centrale di spina. Il conglomerato dei muri esterni, in particolare, è gettato in opera all'interno di casseforme metalliche che imprimono un motivo geometrico astratto, evocando un rivestimento a lastre di pietra. Su questo solido sistema cellulare si impostano poi i dieci pendoli, appuntiti in sommità, che scandiscono il fronte principale e risultano tesi o compressi secondo l'asimmetria dei carichi agenti sulla pensilina – in soletta laterocementizia e sostenuta da mensole a profilo curvilineo – che contribuiscono a stabilizzare.
Visto dalla piazza antistante, il lunghissimo blocco sembra fluttuare sul profondo e schiacciato scuro del piano terreno. È invece sollevato dal suolo grazie a due pilastroni – coadiuvati da puntoni inclinati – caratterizzati dall'iconica sagoma a iperboloide iperbolico a sezioni ellittiche, realizzato con una cassaforma artigianale assemblata in travetti di legno di balsa, a sezione trapezoidale.

salvandone l'impostazione di massima dei volumi. Il corpo della Giunta, su cui si concentrano le sue attenzioni, è inizialmente ipotizzato sostenuto da una trave Vierendeel centrale, che avrebbe sospeso l'edificio grazie ad altre dieci Vierendeel trasversali, proiettate a sbalzo per otto metri su entrambi i fronti. Successivamente, Libera coinvolge Musmeci nella progettazione, con cui apporta ulteriori, decisive modifiche. I lavori sono appaltati nel 1959 e nel 1962 è terminata la costruzione della struttura. Il completamento dell'edificio viene affidato poi dalla ditta Pierino Bonvecchio di Trento. Per la difficoltà a reperire i fondi necessari, quando Libera muore, nel 1963, è ancora in corso la posa di rivestimenti, finestre e vetri, poi conclusa sotto la direzione dell'ufficio tecnico regionale.

Tra l'ardito dispositivo cementizio che sostiene l'edificio e la sobrietà complessiva della sua immagine si genera una singolare tensione, che viene liberata in alcuni punti strategici (Reichlin, 2007): nei bordi inferiori dei setti cementizi che emergono all'intradosso del piano terreno, nella sequenza di travi-parete bucate dalle finestre rettangolari del prospetto, nell'aggetto della pensilina di coronamento sostenuta dai pilastrini e, infine, nei due pilastroni dalla superficie *optical.* L'edificio rappresenta quindi, anche per questo, l'esito maturo delle riflessioni di Libera sugli aspetti figurativi della struttura e, grazie all'apporto di Musmeci, uno dei momenti più significativi della contaminazione, tipica dell'Italia di questi anni, tra il mondo dell'ingegneria, dell'architettura e del design.

Un grattacielo all'italiana in Canada

Le ricerche condotte sulle opere romane di Moretti e sugli edifici alti milanesi hanno stimolato un'ultima indagine, quella sulla torre della Stock Exchange di Montreal. L'edificio era meno noto nella storia dell'architettura italiana rispetto ai precedenti cinque, ma ad attrarre l'attenzione hanno concorso alcuni fattori: è stato il secondo grattacielo realizzato su progetto di Nervi, dopo il successo del Pirelli; pure realizzato all'estero, è stato curato nei minimi dettagli da Moretti, grazie al suo rapporto di fiducia con la società committente – la Generale Immobiliare, la stessa della torre Velasca –, italiana anch'essa; è stato, inoltre, il grattacielo in cemento armato più alto del mondo al momento della sua costruzione, grazie ai suoi 190 metri. Lo studio della documentazione originale – fondamentali, anche stavolta, gli archivi di Pier Luigi Nervi al MAXXI e al CSAC, quelli di Moretti conservati a Roma e quello della SGI presso l'Archivio Centrale di Stato, ma anche l'archivio dell'Ismes e quello, fortunatamente consultabile on line, del Canadian Intellectual Property Office Innovation – ha rivelato particolari poco conosciuti.

Incaricati dalla SGI, Moretti e Nervi[8] sostituiscono un progetto già elabo-

[8] La Stock Exchange Tower di Montreal è progettata da Moretti insieme a Nervi e a un team di architetti e ingegneri nordamericani dalla fine del 1960, quando i due sono scelti dalla SGI, che dirige l'operazione, per elaborare collegialmente il progetto generale. Eserciteranno anche l'"alto controllo" sugli esecutivi architettonici e strutturali (cfr. Place Victoria St. Jacques Co. Inc., *Incarico professionale all'arch. Luigi Moretti e al prof. ing.*

rato, in precedenza, dal gruppo di architetti americano SOM, confezionato in perfetto Stile Internazionale e si confrontano a suon di proposte, presentate alla società committente, per prendere nettamente le distanze da tale modello, ritenuto ormai troppo scontato. Mentre Nervi, però, caratterizza i suoi studi mediante l'uso della struttura portante in chiave linguistica, Moretti elabora soluzioni che si distinguono per la libertà con la quale sono composte le volumetrie.
Nervi, accantonando, come nel Pirelli, gli schemi strutturali tradizionali, organizza lo scheletro di questa torre da record in due distinti dispositivi: il "sistema resistente principale" e un sistema strutturale secondario. Il primo assicura la stabilità complessiva dell'edificio, opponendosi alle azioni orizzontali, ed è composto da un nucleo centrale – costituito da due setti in cemento armato, disposti a X in pianta – collegati da tre serie di travi reticolari ai quattro pilastri disposti ai vertici della pianta, di circa 40 metri di lato. L'altro, costituito da otto pilastri minori, collabora solo a sostenere il peso della batteria dei solai.
La soluzione del sistema principale, alla cui definizione contribuisce anche l'ingegnere canadese Jacek Barbacki, è avveniristica ed anticipa gli schemi ad "outrigger" con cui Fazlur Khan, dalla fine degli anni Sessanta, progetta alcuni dei suoi più importanti grattacieli. In questo modello i pilastri perimetrali, coinvolti nella stabilizzazione alle azioni orizzontali, sono particolarmente efficienti grazie all'elevata distanza dall'asse verticale del fabbricato. Si comportano come pendoli, alternativamente tesi o compressi, e il ragguardevole momento stabilizzante che mettono a disposizione è garantito dal massimo braccio delle forze, conseguente alla geometria dell'edificio, pari alla diagonale della pianta.
Mentre il dimensionamento dell'ossatura procede nei laboratori e sui tavoli degli ingegneri, ai due progettisti italiani non sfugge che l'edificio permette anche una nuova riflessione sul tema dell'espressione della

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Pier Luigi Nervi, firmato il 29 marzo 1961, conservato presso l'Archivio centrale dello Stato – Fondo SGI), della cui elaborazione sono incaricati, rispettivamente, gli studi canadesi Greenspoon, Freedlander & Dunne e D'Allemagne & Barbacki. Durante lo sviluppo della progettazione il complesso, destinato ad ospitare la Borsa valori della città, è ridotto da tre torri disposte lungo la diagonale a due, allineate e collegate da un corpo basso. I lavori sulla prima torre, iniziati nel giugno del 1963 e condotti dalle ditte EGM Cape & Co. e A. Janin & Co., sono portati a termine in poco meno di due anni, mentre l'ipotesi del raddoppio viene progressivamente abbandonata. Moretti studia alcune varianti per la seconda parte del complesso fino al 1972, ma alla fine sul lotto è realizzato un fabbricato in completa difformità rispetto ai suoi progetti iniziali.

struttura nell'edificio alto: per entrambi, ormai, il modello del grattacielo americano non è più soddisfacente. Le strade individuate dall'ingegnere e dall'architetto, però, stavolta non convergono.
Nervi, fedele alla sua poetica, esprimerebbe infatti il sistema strutturale facendolo coincidere con l'immagine, senza alterazioni, fiducioso che la razionale ossatura messa a punto sia sufficiente a garantire una intrinseca qualità formale all'edificio. In questo progetto prevale, però, Moretti, che sottopone lo scheletro e l'intero volume a una serie di correzioni ottiche che ne modificano completamente la percezione (Capurso, 2020).
Alcune sono riconducibili al più ampio alveo della strategia della forma finita già vista nei grattacieli realizzati in Italia, come il progressivo arretramento dei solai, di pochi centimetri per piano, allo scopo di rastremare l'intero volume. Queste correzioni non bastano però all'architetto che, sempre contro il parere di Nervi, ottiene altre modifiche (Capurso, Poretti, 2010).
Per slanciare maggiormente l'immagine della torre, l'altezza delle travi reticolari di controvento è ridotta infatti dal basso verso l'alto, contraddicendo il reale andamento degli sforzi. Gli otto pilastri del sistema strutturale secondario sono occultati dietro il curtain wall bombato, con costosi accorgimenti costruttivi finalizzati anche a non penalizzare la superficie commerciale degli uffici. I quattro elementi d'angolo, infine, sono ingigantiti artificiosamente grazie a una scocca, realizzata con pannelli di cemento bianco, il cui sviluppo in altezza presenta addirittura un'entasi simile a quella delle colonne doriche, per comunicare, come voleva Moretti, la "palese sofferenza" dei pilastri.
L'ultimo ridisegno elaborato per le indagini, dedicato appunto alla torre, ha permesso così di analizzare l'effettivo impatto di queste minuziose scelte – ovviamente incomprensibili per i pragmatici costruttori nordamericani – sulla gigantesca costruzione e, allo stesso tempo, di apprezzare il modo in cui la struttura assume, ancora una volta, un ruolo centrale nell'ineffabile originalità dell'edificio.

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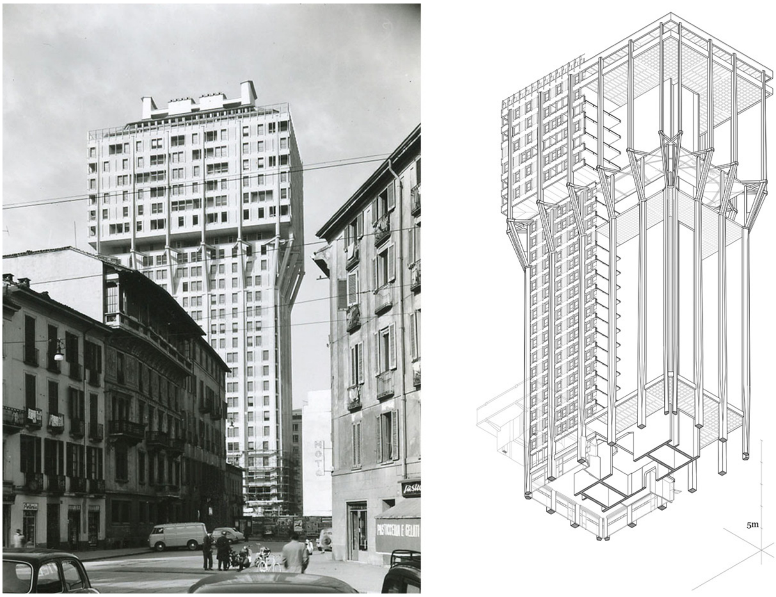

Fig. 1. Torre Velasca a Milano (1950-1957), foto d’epoca (Archivio Fotografico Paolo Monti) e spaccato assonometrico complessivo (disegno dell’autore).

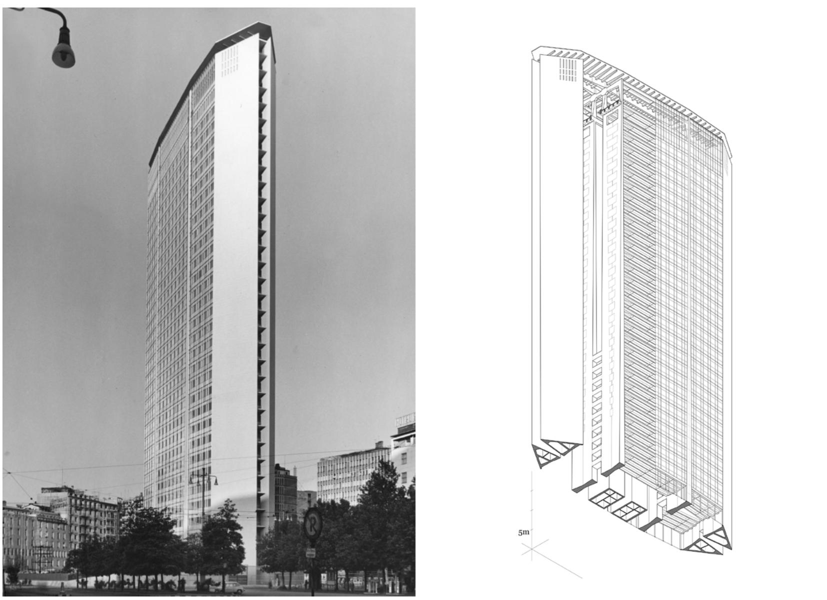

Fig. 2. Grattacielo Pirelli a Milano (1954-1960), foto d’epoca (Archivio Storico Pirelli) e spaccato assonometrico complessivo (disegno dell’autore).

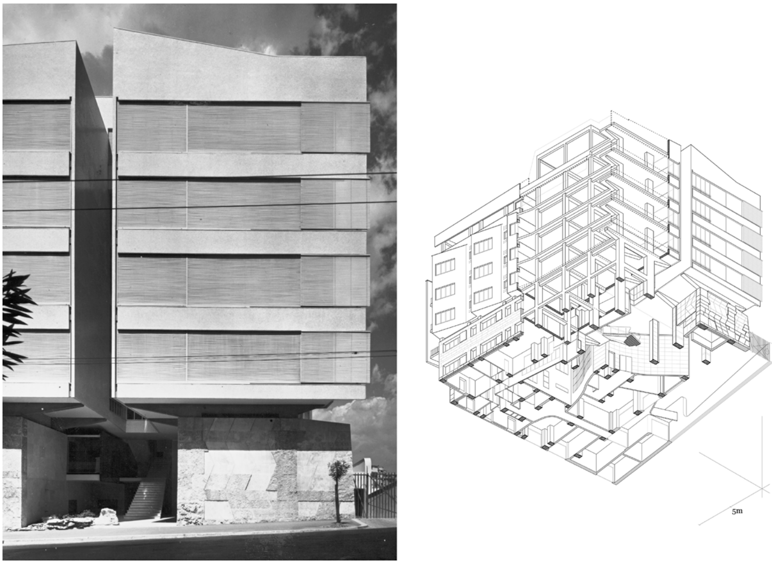

Fig. 3. Palazzina "Il Girasole" a Roma (1947-1951), foto d'epoca (Archivio Centrale dello Stato, Fondo Luigi Moretti, Roma) e spaccato assonometrico complessivo (disegno dell'autore).

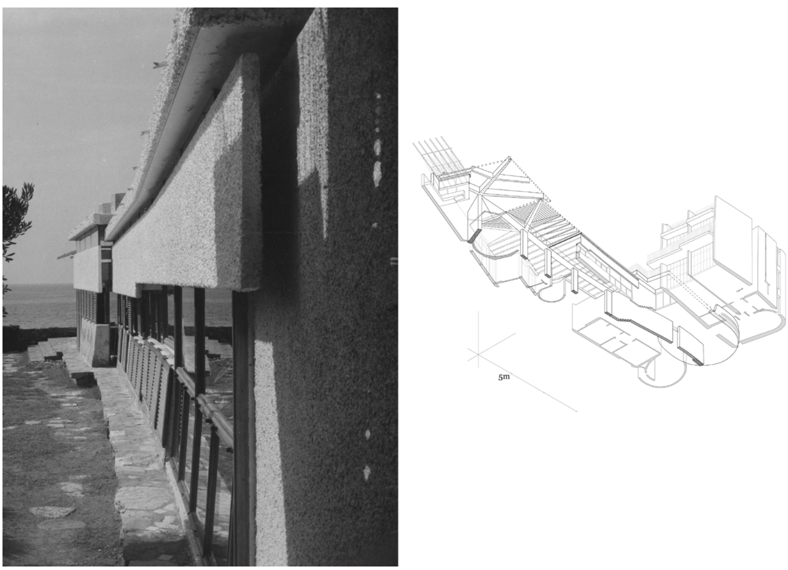

Fig. 4. Villa "La Saracena" a Santa Marinella (1955-1957), foto d'epoca (Archivio Centrale dello Stato, Fondo Luigi Moretti, Roma) e spaccato assonometrico complessivo (disegno dell'autore).

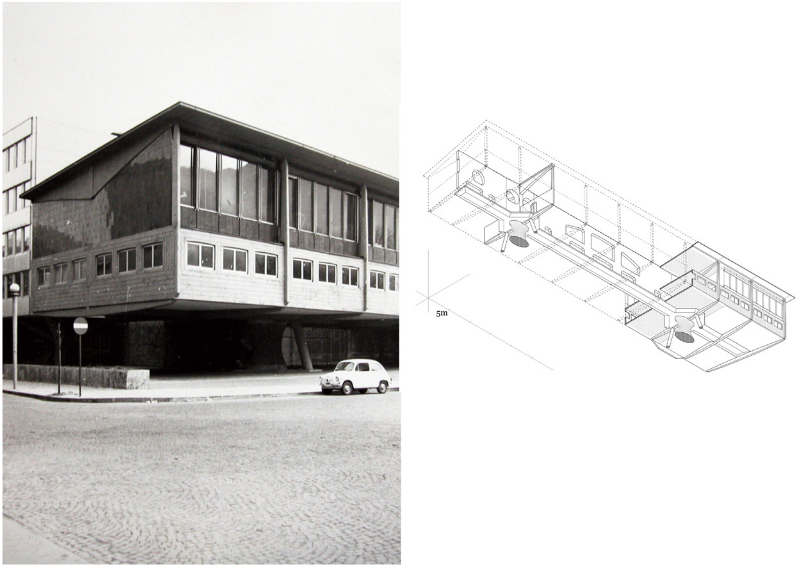

Fig. 5. Palazzo della Regione a Trento (1953-1966), foto d’epoca (Archivio Regione Trentino-Alto Adige – Ufficio Tecnico, Trento) e spaccato assonometrico complessivo (disegno dell’autore).

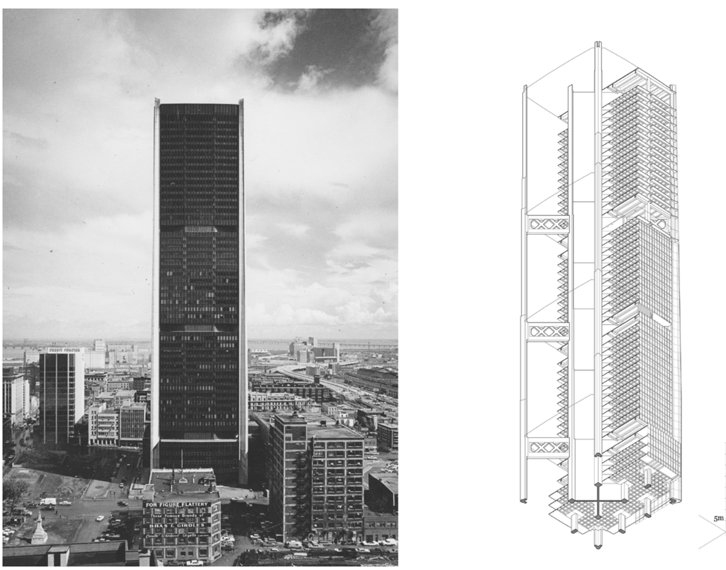

Fig. 6. Stock Exchange Tower a Montreal (1960-1965), foto d’epoca (Archivio Centrale dello Stato, Fondo Società Generale Immobiliare, Roma) e spaccato assonometrico complessivo (disegno dell’autore).

Edoardo Benvenuto Prize
Abstracts

Pierre Smars

Safety analysis of masonry arch and vaults with a note on the kinematic safety of arches

The present paper analyses the influence of large displacements of the abutments on the stability of arch structures. A series of typical deformation scenarios are studied in detail (horizontal spreading, settlement of abutments, complex deformation paths). A phenomenological description is first given based of a large number of experiments on a scale model. Experiments were filmed using a high-speed camera and frames corresponding to key events were extracted for measurements (hinge formation, collapse). A range of behaviours is identified and, in some circumstances, a significant spread in the results is observed. The same scenarios are then studied using a mathematical model. A flexible set of software tools was developed to analyse 3D block structures of arbitrary shapes, subjected to arbitrary deformations (interactively or following preset algorithms). It is used to clarify the process of hinge formation, hinge pattern transitions and the origin of the variations in the results.

Ilaria Pecoraro

I sistemi voltati del Salento: origini, geometrie costruttive e problemi di conservazione

This contribution summarizes years of study and restoration work on the Lecce vaults investigated from a historical-critical and technical-constructive point of view. The research highlights the confinement of this long-lived phenomenon in the province of Terra d'Otranto and interprets it as the peripheral, specialized and standardized remodulation of the Valencian experience in the second half of the fifteenth century. The star vaults follow the constructive logic of isometric rows 'in folio' with a self-supporting cap, with angular details. Built continuously from the 16th to the 20th century, the star vaults are an identifying factor in the history of Salento architecture, worthy of knowledge, protection and conservation.

Tullia Iori

Reinforced concrete in Italy: a sweeping story

Based on previously barely utilized sources from the Patents Office Archives and specialized engineering magazines, this study reconstructs the use of reinforced concrete in construction work in Italy from its first uses up to the Second World War.
How did this new technique begin to spread at the end of the last century? To what extent was this new technique imported from abroad? And how did foreign systems set off its autochthonous development? What role did the post-earthquake reconstruction of Messina and Reggio play in this? Moreover, what role did Italian researchers occupy in the international debate on the adaptation of a classical theory to a non-homogeneous material? And subsequently, how were the limitations placed on the use of steel during Italy's autarchic period overcome? How did the development of reinforced concrete continue? And while this technique became widespread in daily construction work, how did it develop in the experimentation of greater works?

John Ochsendorf

The stability of a masonry arch on buttresses

There are two main classes of masonry structure: arches that thrust, and supporting elements, such as walls and buttresses, which resist the thrust. This paper investigates the stability of a masonry arch supported on buttresses and the conditions necessary for collapse to occur. Masonry structures can be severely distorted over the years, often due to subsidence or long-term movements in the foundations, and this paper provides guidance in the assessment of such structures. The resistance of masonry buttresses to horizontal loads is examined. In the case of failure due to overturning, a fracture will develop in the masonry, significantly reducing the resistance of the buttress. The capacity is further reduced by outward leaning of the buttresses, a common source of distress for masonry structures due to movements in the supporting foundations. Based on these considerations, new measures of safety are proposed for buttresses under horizontal loading. Outward leaning of the buttresses increases the span of the arch or vault. Spreading supports will cause large deformations in the arch, which increase the horizontal thrust of the arch and may lead to collapse. These problems are analysed for circular masonry arches, and the collapse conditions are identified for various geometries. The findings are combined to investigate the stability of the masonry arch supported on buttresses, including a case study of a barrel-vaulted structure in Goa, India. As the buttresses lean, the thrust of the vault increases and the resistance of the buttress decreases. This analysis illustrates that the arch will collapse, and the buttresses will remain standing in most cases.

Joaquín Francisco Antuña

Las estructuras de edificación de Eduardo Torroja Miret

The text presents the results of various initiatives carried out since the presentation of the thesis "The building structures of Eduardo Torroja Miret" that arose as a result of the investigation initiated for the writing of the thesis. They are presented in chronological order, so that it can be seen that the research results have influenced both my professional dedication and the teaching initiatives in which I have participated. The activities have been grouped into six themes of different breadth. The first refers to the knowledge and dissemination of the content of the Eduardo Torroja Archive that the work carried out in the thesis helped to disseminate, also including some research that this information made possible. The second theme is related to collaboration with architects and focuses on Torroja's work with Manuel Sánchez Arcas, author of the project for the dome of the Algeciras market. The third focuses on a work, the roofs of the grandstands of the Zarzuela racetrack. The fourth in the investigation carried out by Torroja to solve the construction of deposits with different techniques and his contribution with his own prestressing system. The fifth refers to the edition of the construction lexicon that Torroja promoted from the Technical Institute of Construction and Cement and, finally, the work carried out by Torroja in the field of experimentation with models.

Chiara Calderini, Chiara Ferrero, Pere Roca

Experimental and numerical response of dry-joint masonry arches subject to large support displacements

In this paper, the response of dry-joint masonry arches to large support displacements was investigated through experimental tests and numerical analyses. A large experimental campaign was performed on a 1:10 small-scale segmental arch subjected to vertical, horizontal, and inclined support displacements. The experimental tests were simulated by adopting a finite element micro-modelling approach, in which the arch was schematized as an assemblage of very stiff voussoirs interacting at no-tension friction interfaces. Experimental and numerical results were compared in terms of collapse mechanisms, hinge position and ultimate displacement capacity. The sensitivity of the numerical predictions to the interface stiffness was also evaluated. Based on the comparison between experimental and numerical results, a strategy to accurately simulate the response of dry-joint masonry arches to large support displacements was proposed.

Holger Eggemann

Simplified design of composite columns based on construction history and the development of design rules

This paper refers to composite columns of structural steel, concrete and reinforcing steel. Three basic types of composite columns can be distinguished, completely encased sections, partially encased sections, and concrete filled hollow sections. The paper presents a simplified method for dimensioning and design of composite columns, based on the construction history and a comparative study of historic building regulations in the United States and Germany. The method can be used for approximate determination of load resistance in early design stages or to make proof of computer calculations. Construction History plays an important role in the development of the proposed method, as several ideas from ancient authors were taken into account to make it as easy as possible. The simplified design method and the study on construction history of composite columns were both part of the author´s doctoral thesis (Eggemann, 2003), which the Eduardo Benvenuto Prize was awarded for in 2005. This text is a slightly revised version of a contribution to the 2nd International Congress on Construction History in Cambridge held in 2006 (Eggemann, 2006).

Hermann Schlimme

Building knowledge encounters emerging natural science: the 'Accademia della Vachia' in Florence, 1661-1662

The present paper examines the Florentine Accademia della Vachia (1661–62), overlooked in the literature until the present author's research on it (published 2006). In exemplary manner, the members of that academy – scientists, mathematicians, architects, engineers and artists – applied the approach of the emerging modern natural sciences to problems of building technology. The focus of the paper is the construction of a new roof for the church San Giovannino in Florence, which had to be done with innovative trusses without tie beams. The academicians, including those among them without building expertise, developed solutions for the roof trusses and the most convincing idea was applied. The academicians explained their solutions and the respective mechanical bases in the commentaries to their proposals. This episode enables us to trace the process by which a technical solution was developed and adopted in the debate of the period. We see an environment of open and public debate, outside the institutions and their secrets, animated by the desire to apply the approaches of modern science and involving all the experience and knowledge of all who were present, in investigating the reasons that roof constructions behave as they do. The paper also traces how the author continued to investigate both the application of modern science to architectural enterprises in Early Modern time and the continuing importance of practical knowledge and experience of craftsmen in 19th and 20th centuries.

Luc Tamboréro

La coupe des voûtes à la française

The study of the vaults of the town hall of Arles built in 1675, presents a particular stereotomic case difficult to achieve by current stereotomic models, the stereotomy of the vaults becomes the subject of a hybrid geometry hypothesis between the known methods employed by carpenters and stonemasons. We explore in depth the geometric families of geometry applied to construction and situate them in a chronology. This geometric work guides us through the technical archives of the 17th century in search of comparable cases. The discovery of a collection of archives from the Royal Academy of Architecture in the old collection of the Ecole Nationale des Ponts et Chaussées validates the geometric method imagined for the vault of the town hall of Arles. Indeed, Philippe de la Hire, director of the Royal Academy of Architecture, uses a specific stereotomic method which corresponds to our hypothesis from 1688. The study of the genesis of this method finally reveals its social role in the struggles for the professional hegemony. Geometric innovations are placed in the socio-historical context that allows them to see the light of day. A remarkable fact emerges: innovation is generally linked to a change in the balance of power between decision-makers and performers. The conquest of markets regularly involves a scientific conflict with social repercussions.

Denis Zastavni

Robert Maillart's essential contribution to structural design with concrete. Morphogenesis for architectural structures

Swiss engineer Robert Maillart (1872-1940) designed visually striking structures remarkable for their efficiency. For many years though his methods were considered fanciful and approximate. Following work initiated by David Billington in the 1970s, current research shows that his approach is to be considered as an elaborate method leading to exceptional designs. By first examining the clues of the structural problem, Maillart devised the adapted structural response (behaviours) and then studied how to implement it, resulting in concrete being considered differently depending on the structural mechanisms used. As a tool, Maillart turned to Graphic Statics first to define the typology, then to adjust the load path and subsequently to control behaviour by localised dimensioning. In parallel, he went on to assemble the various constituent structural mechanisms into the final arrangement, it being obvious that the choice of appropriate mechanisms depends on the context of the structural problem, particularly the site, leading to differentiated and contextualised designs each time.

Federica Ottoni

Of domes and their trap. The long history of the domed structures, between debates and experimentation

The aim of this paper is to show the potentiality of empiricism in identifying the real mechanical behavior of ancient masonry domes, definitely stating the solution for their primeval "trap".
The mechanical behavior of the domes – their lack of stability due to the horizontal thrusts generated by the arches which compose them – couldn't be mathematically demonstrated by ancient architects but some fundamental historic debates represent as many enhancements in their structural conception.
Starting from the two "*Mecanicopoioi*" of Hagia Sophia, passing through Brunelleschi's terrific inventions, up to the slender design of Sainte Genevieve in Paris, this paper analyzes the issue of the "dome's trap" (its ineradicable horizontal thrusts). The final aim, in an ideal route from the empiricism to the scientific approach, is to clarify the structural behavior of masonry domes, demonstrating the efficacy of the traditional encircling tie-rods intervention.

Marzia Marandola

Autobiografia della ricerca

The essay traces the contents and the role that my book, awarded with a mention at the 2010 Edoardo Benvenuto International Prize, had in my academic career as a researcher and professor of history of contemporary architecture in the faculties of Engineering and Architecture in Italy. The research investigated in the volume focuses on the different construction methods recurring in prestressed works, their static functioning and the design outcomes that characterize some masterpieces of construction in Italy. The story begins with the birth of prestressed reinforced concrete and follows its diffusion in Italy, without submitting to the chronological sequence, but giving voice and figure to the technical and expressive potential that this new material has offered to the construction world.
At the basis of this study is my doctoral research dedicated to engineer Riccardo Morandi (1902-1989): the experiments and works in prestressed reinforced concrete of the fifties, which deals with pioneering research on the construction of Italian experimental works of a large engineer, protagonist of the history of architecture and the history of engineering, designer of some of the most innovative works of the twentieth century.

Ricardo Maia Avelino, Tom Van Mele, Philippe Block

Advances in Thrust Network Analysis. Constrained equilibrium assessment of masonry vaulted structures

In this overview paper, the formulation of Thrust Network Analysis (TNA) is revisited with a focus on its application to the assessment of vaulted masonry structures. TNA offers a fast and flexible methodology to compute lower-bound, admissible equilibrium solutions for a given masonry envelope. The internal forces are discretised in a network with compressive-only axial forces along the edges and external loads and supports assigned to the vertices. When only gravitational loading is considered, the method can be approached using graphic statics offering an intuitive and interactive workflow. A numerical, and more robust formulation is presented, which allows for framing TNA in optimisation processes necessary to its application in a masonry assessment context. In this numerical formulation, the projection of the thrust network is kept fixed in plan, which brings an additional control for the designer to highlight, and follow, major geometrical features of the masonry structure to assess, such as creases, cracks or point loads. New developments of the method are presented focusing on a flexible optimisation framework allowing for multiple objective functions. These objectives focus on providing a consistent measure of the level of stability in the masonry structures and on investigating its cracking pattern through coupling TNA with an energy-based criterion. The presented contribution illustrates the recent advances through several examples showing its wide applicability for lower-bound equilibrium analysis. Finally, the present state-of-the-art of TNA allows it to be used as a practical assessment tool benefiting researchers and practicing engineers in the field of structural preservation.

Stefano De Santis, Gianmarco de Felice

Load-carrying capacity and seismic behaviour of masonry arch bridges

The road and rail infrastructural network in Europe includes a large number of masonry arch bridges. Due to increasing traffic demand and exposure to earthquakes, their safety level needs to be assessed, and it is of the utmost importance that their structural behaviour is understood in depth and that reliable methods for structural analysis are developed that are suitable for engineering practice. This work provides an overview of a PhD research carried at Roma Tre University, which aimed at contributing to the knowledge of masonry arch bridges and proposing a modelling approach for their assessment under traffic and seismic loads. First, the mechanical behaviour of historic brickwork under eccentric compression was investigated in the laboratory. Then, a fibre beam model was calibrated on the basis of test outcomes and used for the structural assessment of masonry arch bridges. A sample of 50 historic rail bridges was selected and their safety level under traffic loads was estimated, also discussing the effect of constitutive assumptions. Finally, the seismic performance of masonry arch bridges was investigated through push-over and non-linear dynamic analyses.

Christian Kayser

Freiburg and the consequences.
Construction and reception gothic tracery spires

Completed in the early 14th century, Freiburg minster's western belfry served as the unsurpassed model for a series of towers in several european regions. This remarkable feat of planning and execution is a pioneer building in several aspects: It introduces an impressive octagonal hall as well as a complex vertical staircase, and it is crowned by the first – and largest – openwork spire. During the building process of the spire, the Freiburg master masons developed an economic and structurally sound building system consisting of only a limited number of standardized elements, i.e. tracery panels, corner struts, crockets and wrought iron anchors. In terms of structure and load transfer, these elements act together as a kind of lattice framework, with the tracery elements taking up compressive forces and the wrought iron tie roads acting as tensile members. The same skillful application of iron elements can also be found at the construction of the tower's crowning finial, with the reinforcement of a vertical iron skewer. In course of the following centuries, about 50 further openwork spires could be implemented. The typology was "exported" along the main trading routes of central Europe along the Rhine and the Danube; travelling masons also brought it to Bolzano in the Central Alps or to northern Spain.

Barbara Berger

Gasholder construction. A multi-disciplinary history

Before the rising structures of gasholders changed the cityscapes, it was the gaslight itself, that was revolutionizing daily life in the cities in the beginning of the 19th century: in the end of 1813 public illumination from gas was introduced for the very first time in Westminster, London.
Gasholders are technical buildings that were constructed to store locally produced coal gas for lighting in the 19th and early 20th century. These emerging iron structures presented a new kind of industrial architecture and became a symbol for the gas industry. The function of the gasholder determined its structure and was initially built with a water-sealed system composed of a water tank, a guide frame and lifts. Over the 19th century the development advanced from the bell-type and telescope-type gasholders (linear and spiral-guided), to the waterless or dry gasholders (piston-type gasholders). Thus a building type emerged, which represents the result of the interplay of various disciplines: technology, engineering and architecture. In the 1960s coal gas was gradually replaced by natural gas and new storage systems. Today, the historic gasholders are industrial relics, many of them demolished. This paper sums up the long centenary history of the gasholder, its significance, development, form and structure as a part of engineering and industrial heritage.

Gianluca Capurso

Struttura e architettura. Strumenti e risultati di una ricerca sul dopoguerra italiano

Post-war Italian architecture was strongly influenced by structure, even when it did not have to deal with challenging static problems. Structure entered the architectural language, generated it, modified it, whether in the Roman palazzina or in the Milanese skyscrapers, in the serious institutional buildings but also in the villas by the sea. The research work on "Structure and Architecture. Investigations on post-war Italy", which received the special mention of the 2019 edition of the Edoardo Benvenuto Prize, focuses on six famous works born from the collaboration between famous Italian architects and engineers: Pier Luigi Nervi, Sergio Musmeci, Arturo Danusso, the lesser known Silio Italico Colombini with Adalberto Libera, Luigi Moretti, Gio Ponti, the BBPR. The works and, in particular, their reinforced concrete structures – exhibited or hidden, real or ideal – have been subjected to an anatomical analysis that reveals different declinations of what Sergio Poretti called the " Italian Modernisms".
This contribution retraces the development of the study conducted by the author, also dwelling on operational and methodological aspects, such as the research in the archives and the elaboration of the "reconstructive redrawings" with which the works were analysed, and summarises the most significant results.

Edoardo Benvenuto Prize
Biographies

Joaquín Francisco Antuña

Architect since 1989 from the Universidad Politécnica de Madrid and doctor of architecture since 2003 from the same university. Professional activity from 1989 to 2010 and structural consultant since 2010. Associate professor from 2006 to 2010 and tenured professor from 2010 to the present. Works of architecture awarded at the Spanish Architecture Biennials of 1997 (IV), 1999 (V), 2005 (VIII), 2008 (X) and 2016 (XIII). Exhibition of works in which I participated collaborating with the architect at the Museum of Modern Art in New York 2006. Member of the executive council of the International Association of Structures and Architecture (ICSA) since 2019. Coordinator of the doctoral program in building structures between 2015 and 2018 and secretary of the same from 2018 to date. Secretary of the Academic Committee of the Master's Degree in Building Structures of the Higher Technical School of Architecture of Madrid. Investigation in intervention in historical buildings with reinforced concrete structures 2008 until now. Research in cold-formed profiles (Light-gauge steel) from 2010 to the present.

Barbara Berger

Barbara Berger studied architecture at the Technical University of Munich and the Istituto Universitario di Architettura in Venice. After her diploma on the Venetian gasholders (2009) she practiced as an architect in the engineering studio Barthel & Maus in Munich and Mainz (today Kayser + Böttges), working on projects in the field of preservation of monuments until 2012. She took the doctoral degree with the dissertation on "Der Gasbehälter als Bautypus" (supervised by Prof. Manfred Schuller and Prof. Rainer Barthel, TU Munich) in 2019 for which she was awarded international grants (German Study Center Venice and German Historical Institute London) and prizes. During her PhD she worked as an architect for the engineering studio Haushochdrei, Munich and in 2017 changed to a lecturing position at TU Munich for the study program "Vocational School Teacher – Building Technology". Since 2020 she is working at the ETH Zurich: until 2021 in a postdoc-position at the chair of "Construction Heritage and Preservation", Prof. Silke Langenberg and since 2022 as a lecturer for the chair of "History of Technology", Prof. David Gugerli. In 2022 she started her own company "Industriekultur und Architektur", based in Zurich.

Philippe Block

Prof. Dr. Philippe Block is professor at the Institute of Technology in Architecture (ITA), ETH Zürich, where he leads the Block Research Group (BRG) with Dr. Tom Van Mele and is Head of the Institute. Philippe is also Director of the Swiss National Centre of Competence in Research (NCCR) in Digital Fabrication. He studied architecture and structural engineering at the Vrije Universiteit Brussel and at the Massachusetts Institute of Technology, where he under the guidance of Prof. John Ochsendorf earned his PhD in 2009. His dissertation, titled “Thrust Network Analysis: Exploring Three-dimensional Equilibrium,” was awarded the Edoardo Benvenuto Prize in 2012.

Chiara Calderini

Chiara Calderini is Associate Professor of Structural Engineering at the Department of Civil, Chemical and Environmental Engineering of the University of Genoa. Her research is oriented to the structural analysis and experimental testing of historic structures, with focus on hydro-geological and seismic risks. She is the scientific Responsible of the “Laboratory of Material and Structures” of the University of Genoa and Head of the Master Program in “Engineering for Building Retrofitting”. She is member of the Editorial Board of different scientific journals, such as “International Journal of Architectural Heritage”, “Buildings”, “Frontiers in Built Environment”. She has lead many research projects, funded by the European Commission and the Italian Ministry of Research and University. She has been consultant of the Italian Ministry of Cultural Heritage in many case studies on historical buildings. She was the Winner of the International “Edoardo Benvenuto” Prize for the PhD thesis “A constitutive model for masonry: formulation and implementation for the analysis of complex structures” in 2004.

Gianluca Capurso

Gianluca Capurso has a degree in Building Engineering - Architecture and a PhD in “Building Engineering: Architecture and Construction” from the University of Rome Tor Vergata.

Since 2005 he has participated in the research group “Architecture and Construction” coordinated by Sergio Poretti and Tullia Iori. In the field of construction history, in particular, he focuses his studies on the relationship between structure and architectural language in Italy from the Second World War to the end of the twentieth century. In the same group he also participates in activities in the field of conservation and valorisation of modern architecture. Since 2013 he is Senior Researcher of the project SIXXI - “XX Century Structural Engineering: the Italian Contribution” at the University of Rome “Tor Vergata” (www.sixxi.eu) within which he deals with: the evolution of automatic calculation methods applied to structural engineering, with particular regard to the experiments carried out on the subject by Italian engineers; the transformations of the Italian School of Engineering since the 1960s, also in relation to the socio-economic transformations of the country after the boom; the achievements of Italian engineers and firms abroad in the second half of the 20th century.

Gianmarco de Felice

Gianmarco de Felice is Full Professor of Construction Techniques at the Engineering Department of the Roma Tre University. He was chairman of the RILEM TC-250 CSM Technical Committee; of the ACI 5490L Committee, member of the Management Committee of the COST TUD 1207 Action; member of the drafting committee of the “Charter of Rome on the resilience of art cities to natural disasters”; associate editor of the Frontiers Structural Materials Journal and the Journal Advances in Civil Engineering. He was scientific coordinator for heritage conservation and structural rehabilitation projects: the restoration of the Abbey of San Clemente in Casauria, financed by the World Monuments Fund and awarded by the Domus International Award for Restoration and Conservation, the seismic upgrading of the Convent of San Bernardino after the 2009 LAquila earthquake, the restoration of Palazzo Farnese in Ischia di Castro. His current research activity includes structural behavior of architectural heritage, seismic safety assessment and retrofitting of masonry and reinforced concrete structures, soil-structure interaction modeling, development of composite reinforcement systems, mechanical characterization of composites. He is responsible for various research projects on these topics and has been invited to give talks at various conferences, including 8th SAHC 2012, 15th MASE 2013; MuRiCo4 2014; Academy of the Lincei 2015; 10th SAHC 2016.

Stefano De Santis

Stefano De Santis is associate professor of Structural Engineering (Tecnica delle Costruzioni) at the Department of Civil, Computer and Aeronautical Engineering of Roma Tre University, in Rome, Italy. He is a member of the Structures Research Group, of the Scientific Board of the PhD School in Civil Engineering, and teaches Design of Steel and Reinforced Concrete Structures (BSc) and Special Structures (MSc). He got his BSc, MSc and PhD in Civil Engineering at Roma Tre University. Before getting his current position, he was a researcher at the University of the West of England at Bristol, UK (2012) and at Roma Tre University (2013-2022). Stefano's scientific interests include laboratory and field testing of traditional and innovative materials and of full-scale structural members, strengthening with composites, seismic assessment of existing constructions, including bridges and architectural heritage, and innovative techniques for laboratory testing and structural health monitoring. On these topics, he is author of more than 100 scientific papers, supervisor of MSc and PhD theses, and contributor to national and international research projects and technical committees worldwide.

Holger Eggemann

Holger Eggemann studied Mathematics at Cologne University and Civil Engineering at RWTH Aachen. He holds a Dipl.-Ing. and a doctorate from the RWTH Aachen. During his post doctoral studies at the Institute of Structural Design at the Faculty of Architecture he worked together with Karl-Eugen Kurrer on the construction history of bridges—namely the "Melan-System". Soon after his post doctoral studies he worked for h+p ingenieure in structural engineering for several thriving years. Afterwards, he has been a successful Programme Director for Civil Engineering, Architecture and Mechanical Engineering with the Deutsche Forschungsgemeinschaft DFG since 2009. This includes advicing scientists and scholars during the application process, organizing and moderating the review of proposals and also supervision of funded research projects in all DFG programmes such as collaborative research centres, research groups, priority programmes and numerous individual grants.

Chiara Ferrero

Chiara Ferrero is a postdoctoral fellow at the Department of Civil, Chemical and Environmental Engineering of the University of Genoa (Italy). In 2012, she graduated in Building Engineering and Architecture at the University of Genoa. After graduation, she worked for some years as a freelance engineer in structural analysis, seismic assessment, and strengthening of existing masonry buildings. In 2016/2017, she attended the Erasmus Mundus Master course in Structural Analysis of Monuments and Historical Constructions (SAHC). In 2021, she completed her PhD with a dissertation entitled "Structural behaviour of masonry arches on moving supports: from on-site observation to experimental and numerical analysis". Her PhD studies were carried out in cotutelle between the University of Genoa and the Polytechnic University of Catalonia (Spain). Her research interests include experimental testing, numerical modelling, and structural analysis of existing masonry buildings, with a focus on built cultural heritage. Her current research is mainly devoted to investigating the effects of foundation settlements and landslide phenomena on the behaviour of arched and vaulted masonry structures.

Tullia Iori

Tullia Iori is an historian of structural engineer, with research interests including the history of construction and the conservation of modern architecture.Trained in Italy, she received her PhD in 1999. She became Research professor (1999), Assistant professor (2001), Associate professor (2005) and then Full professor (2013) in Rome Tor Vergata University. She is co-Investigator in the SIXXI project (ERC Advanced Grant - P.I.: Sergio Poretti; see also: www.sixxi.eu). Her most recent monographs include: *SIXXI. Storia dell'ingegneria strutturale in Italia*, Voll. 1-5 (Gangemi, 2014-2020); *Visioni strutturali* (ed, Quodlibet, 2022); *Pier Luigi Nervi. L'Ambasciata d'Italia a Brasilia* (with S. Poretti, Electa 2018); *La Scuola italiana di Ingegneria* (eds, with S. Poretti, Quodlibet, 2016). She curated exhibitions at MAXXI Museum in Roma on Sergio Musmeci (2022-23) and on Pier Luigi Nervi (2021 and 2011).
See also: www.tulliaiori.com

Christian Kayser

Christian Kayser (1980) studied Architecture at the TU München and the University of Bath, with a focus on historic constructions. After graduating, he worked as a building archaeologist on the hellenistic sito of Olba/ Diokaisareia in modern-day Turkey. From 2004 on, Kayser was employed as architect at the renowned engineering company Barthel & Maus, Beratende Ingenieure, an expert office in the field of analysing and safeguarding historic structures. In this office, he rose the rank of CEO (2012); since 2019 he is managing partner of the company (now: Kayser+Böttges, Barthel+Maus, Ingenieure und Architekten GmbH).
Parallel, Christian Kayser worked 2008-2011 as assistant professor at the chair of Prof. Rainer Barthel (Structural Engineering) at the TU München, and finished his doctoral thesis on the construction of gothic window tracery, which was awarded the Premio Eduardo Benvenuto XI. Since 2012, Kayser is lecturer at the TUM as well as the Ludwig-Maximilians-Universität München. 2023 he completed his habilitation treatise on the construction of gothic openwork spires at the TUM.

Ricardo Maia Avelino

Dr. Ricardo Maia Avelino is a postdoctoral researcher at the Block Research Group, ETH Zürich. Ricardo's work focuses on developing novel computational analysis methods to assess complex unreinforced masonry structures and designing efficient structural systems. Ricardo defended his PhD at ETH Zürich in December 2022, being nominated for the ETH Medal for outstanding dissertation. Before joining ETH, Ricardo obtained his Master's in Structural Engineering at École des Ponts et Chaussées, being awarded the Prix Betancourt, and his Bachelor's in Civil Engineering at Escola Politécnica da Universidade de São Paulo.

Marzia Marandola

Marzia Marandola (Rome 1975) is PhD (2006) in Architecture and Construction at the University of Rome Tor Vergata, with a thesis on the work and archive of Riccardo Morandi. From 2011 to 2021 she was a university researcher of History of Architecture at the Department of History, Design

and Restoration of Architecture of Sapienza University of Rome. Since 1 March 2021 she has been an associate professor at the IUAV University of Venice.
Since 2004 she has carried out research investigating the architecture of the twentieth century and in particular the constructive aspects of the works of the twentieth century, with particular regard to the birth and development of the various construction systems and the experiments of Italian engineers. She is responsible for the architectural history research unit for the funded project “The Getty Foundation - Keeping It Modern Program 2018” The Gio Ponti School of Mathematics in Rome (Head S. Salvo).
She has held seminars and conferences in Italian and foreign universities and has published numerous writings on twentieth-century architecture; she regularly collaborates with Italian and international magazines: “Casabella”, “Arketipo”, “EDA. Examples of Architecture”; she is part of the editorial board of the journal “Annali delle Arti e degli Archivi. Painting Sculpture Architecture” of the National Academy of San Luca. Since 2021 she has been an effective member of the Italian National Committee of ICOMOS.

John Ochsendorf

John Ochsendorf is the Class of 1942 Professor of Architecture and Civil and Environmental Engineering at MIT, where he has served on the faculty since 2002. He earned his Bachelor of Science in structural engineering and archaeology from Cornell University, his Master of Science in Civil and Environmental Engineering from Princeton University and his PhD in structural engineering from the University of Cambridge, where he studied with Professors Jacques Heyman and Christopher Calladine. He is a founding partner of Ochsendorf DeJong and Block LLC, a consulting firm specializing in masonry structures.
Ochsendorf has won numerous awards for research in structural engineering and architecture, including a Graduate Research Fellowship from the National Science Foundation, a Fulbright Pre-Doctoral Scholarship from the J. William Fulbright Foundation, a Rome Prize from the American Academy in Rome, and a MacArthur Fellowship from the John D. and Catherine T. MacArthur Foundation.
He is the author of *Guastavino Vaulting: The art of structural tile* (Princeton Architectural Press, 2010) and served as lead curator for a public exhibition on the work of the Guastavino family which toured from 2012-2014.

Ochsendorf contributed to the design of numerous award-winning structures, including the Mapungubwe Interpretive Centre and the Sean Collier Memorial. From 2017-2020, Ochsendorf directed the American Academy in Rome.

Federica Ottoni

Federica Ottoni (Parma, 1977), Engineer, Associate Professor in Restoration at the University of Parma, where she has been teaching since 2010 in the Architecture and Civil Engineering courses. Since 2020 she has been a lecturer in charge of the module "Monitoring and evaluation of existing structures" at the SSBAP of the Milan Polytechnic.
Since 2012 he has been a member of the Italian Society of Architectural Restoration (SIRA - CD 2012 - 2015); he is a member of DO.CO.MO.MO Italia (since 2020), Construction History Society (since 2016) and ICOMOS Italia (since 2023).
Author of numerous national and international publications, her main research interest is the study of instability of historic buildings, in relation to dimensional principles, with particular reference to the theory of equilibrium combined with structural analysis. Among others, it has dealt with the stability and conservation of Santa Maria del Quartiere in Parma, the Roman amphitheater of Durres (Albania), the Citadel of Damascus (Syria) and the domes of the Madonna dell'Umiltà (Pistoia) and of Santa Maria del Fiore (Florence). She is the author of a monograph on the stability of historic domes (Edoardo Benvenuto Prize 2009) and of a volume on proportional principles in masonry (Scientia abscondita, Marsilio, 2019) and of several articles in national and international journals. Director of the MADLab editorial series (Quasar Editore) and Head of MADLab - Laboratory for Monitoring, Analysis and Diagnostics of the Built (DIA) - https://www.madlab.unipr.it/.

Ilaria Pecoraro

Ilaria Pecoraro is an architect, specialist in restoration of architectural monuments and landscape and PhD in Restoration of Architectural Heritage at 'Sapienza', University of Rome, Faculty of Architecture. He lives and works between Puglia and Lazio, dedicating himself to study and research

activities, applied in experimental sites for the restoration of monumental and rural heritage.
For 25 years he has also dedicated himself to the training of future architects, at the Master's Degree Course in Architecture/Restoration, in Italian and in English, holding the Consolidation course and organizing the International Summer School on historic building techniques in Ostuni. Consultant to public administrations and dioceses for the protection of the landscape and the energy efficiency of historic centres, he also designs and builds Architecture of the Faith. Participates enthusiastically in international research and conferences, publishing scientific and technical-dissemination contributions specialized in architectural and static restoration of monumental and historical-archaeological emergencies.

Pere Roca

Pere Roca is Full Professor at the School of Civil Engineering of Barcelona in the Technical University of Catalonia (UPC) where he carries out his activity as lecturer, researcher and consultant on building structures. His research is oriented to structural analysis techniques, experimental analysis, masonry mechanics, structural monitoring, seismic evaluation and strengthening techniques with focus on historical structures. He is a member of the ICOMOS International Scientific Committee on Analysis and Restoration of Structures of Architectural Heritage (ISCARSAH), which he chaired during 2005-2008. He is co-editor of the International Journal of Architectural Heritage since 2007. He was the creator in 1995 of the international conference series on Structural Analysis of Historical Constructions and has acted as co-organizer of several of its editions, including the 12th one organized in 2021. He has been consultant in over 100 case studies of existing and historical structures, including Romanesque and Gothic churches and cathedrals, medieval bridges and Modernist buildings. He has collaborated in the study of 9 UNESCO World Heritage buildings.

Hermann Schlimme

Hermann Schlimme studied Architecture at the Technische Universität Braunschweig and at the Università degli Studi di Firenze. PhD History of Architecture, TU Braunschweig 1998. 2002-2007 scientific direction of

the research program “Epistemic History of Architecture” at the Bibliotheca Hertziana, Max Planck Institute for Art History, Rome (cooperation with the Max Planck Institute for the History of Science, Berlin). Edoardo Benvenuto Prize 2006. 2007-2016 Research Associate at the Bibliotheca Hertziana. Co-editor of *Construction History. International Journal of the Construction History Society* (peer-reviewed). Member of the Scientific Committees of the “International Congresses on Construction History” (Cambridge 2006, Cottbus 2009, Paris 2012, Chicago 2015, Brussels 2018, Lisbon 2021, Zurich 2024). Research project on the Western Buildings in the Old Summer Palace Yuanmingyuan in Beijing, cooperation with Beijing Tsinghua Institute for Digitization (THID). 2015 habilitation (right to lecture/*venia docendi*) at the Technische Universität Wien. 2014-2016 Guest Professor at the TU Wien. Since 2017 Full Professor and Chair of History of Architecture and Urbanism at Technische Universität Berlin.
Schlimme’s research focuses on the history of architecture in Early Modern Italy, China-Europe, Epistemic History of Architecture, Construction History, History of 20th and 21st Century Architecture.

Pierre Smars

Pierre Smars is Assistant Professor at the Department and Graduate School of Cultural Heritage Conservation at the National Yunlin University of Science & Technology in Taiwan. He has a degree in architectural engineering from the Catholic University of Louvain, UCL (1989) and completed a master degree in conservation of monuments (1992) and a Ph.D. in engineering (2000), both at the Catholic University of Louvain KUL.
His research interests are the stability of historic structures, surveying techniques and ancient construction techniques. He has a particular interest for Gothic vaults and other objects where geometrical, technical and structural aspects are intimately related. In his Ph.D. dissertation, he studied the influence of the release of the classical plasticity theory hypotheses on the stability of vaulted structures.

Luc Tamboréro

Luc Tamboréro, Ceo of Ateliers Romeo, stonemason of the Association Ouvrière des Compagnons du Devoir du Tour de France (AOCDTF), associate researcher at the Geometry Structure Architecture (GSA) laboratory - ENSA Paris-Malaquais (2004-2016), graduate from EHESS (2008), works on geometric applications in construction. He published his first articles in the proceedings of the international congresses on the history of construction (Madrid, Cambridge) on the vault of the town hall of Arles and on the Saint-Gilles screw. He contributed to the Encyclopedia of Trades of the Compagnons du Devoir, Masonry and stone cutting.

Tom Van Mele

Dr. Tom Van Mele is senior scientist and co-director of the Block Research Group, ETH Zurich, and lead developer of COMPAS, an open-source computational framework for research and collaboration in architecture, engineering, and construction. Tom studied architecture and structural engineering at the Vrije Universiteit Brussel, where he received his PhD in 2008. His technical and computational developments form the backbone of multiple flagship projects, including the Armadillo Vault (2016), Striatus Bridge (2021), and the NEST HiLo unit (2021).

Denis Zastavni

Denis Zastavni is a Dr. Arch. Eng. and Associate Professor in the Faculty of Architecture, Architectural Engineering and Urbanism [LOCI] at the Université Catholique de Louvain in Louvain-la-Neuve, Belgium where he teaches structures, materials and technologies in architecture. He has worked as a structural engineer and architect for over ten years and still works as consulting engineer on specific structures. His doctoral thesis was on Robert Maillart's design methods for which he received the Benvenuto prize in 2008. His main publications are on structural design, pedagogical approaches to teaching structure and technology, and Robert Maillart's designs. He has a passion for research focused on structural design approaches and tools and on deepening his knowledge of Robert Maillart's structural methods. With younger colleagues, he is currently applying these methods to masonry, concrete and timber structures and incorporating them into computer-aided design tools.

Edoardo Benvenuto Prize
Hall of fame

First edition – Year 2002

Jury:

Alfredo Corsanego (Università di Genova)
Salvatore D'Agostino (Università di Napoli)
Anna Sinopoli (Università di Roma La Sapienza)

Winner:

Pierre Smars, *Etudes sur la stabilité des arcs et voûtes. Confrontation des méthodes de l'analyse limite aux voûtes gothiques en Brabant* (Ph.D. Thesis in Docteur en Sciences appliquées; Promoteur: Prof. S. Di Pasquale, Prof. G. De Roeck).

Honorable mentions:

Bruna Gaino, *Du tensor de W.Voigt au sistema covariante de G. Ricci: différentes origines pour un même concept* (Ph.D. Thesis in Docteur en Sciences; Promoteur: Prof. P. Radelet).

Gema Mercedes Lopez Manzanares, *Estabilidad y construcción de cúpulas de fábrica: el nascimento de la téoria y su relación con la práctica* (Ph.D. Thesis, Escuela tecnica de Arquitectura de Madrid, Madrid 1998; Supervisor: Prof. S. Huerta).

Ilaria Pecoraro*, I sistemi voltati nel Salento fra XVI e XVIII secolo: origini, geometria costruttiva e problemi di conservazione* (Tesi di Dottorato in "Conservazione dei Beni Architettonici"; Relatore: Prof. G. Carbonara).

Second edition – Year 2003

Jury:

Luigi Gambarotta (Università di Genova)
Karl-Eugen Kurrer (Ernst & Sohn – Stahlbau, Berlin)
Patricia Radelet de Grave (Université catholique de Louvain)

Ex-aequo winners:

Tullia Iori, *Il cemento armato in Italia dalle origini alla seconda guerra mondiale*, Edilstampa, Roma 2001.

John A. Ochsendorf, *Collapse of masonry structures*, Ph.D. Thesis, June 2002, University of Cambridge, UK; Supervisors: Prof. Jacques Heyman (University of Cambridge), Prof. Cristopher R. Calladine (University of Cambridge)

Honorable mention:

Snezana Lawrence, *Geometry of Architecture and Freemasonry in 19th century England*, Ph.D. Thesis, 2002, History of Mathematics, Open University, U.K.; Supervisor: Prof. Jeremy Gray

Third edition – Year 2004

Jury:

Rolf Gerhardt (RWTH Aachen)
Riccardo Gulli (Alma Mater Studiorum - Università di Bologna)
Santiago Huerta (Universidad Politécnica de Madrid)

Ex-aequo winners:

Joaquín Antuña Bernardo, *Las estructuras de edificación de Eduardo Torroja* (Tesis Doctoral, Universidad Politécnica de Madrid, Escuela Técnica Superior de Arquitectura, Tutor: Prof. Ricardo Aroca Hernández-Ros).

Chiara Calderini, *Un modello costitutivo per la muratura: formulazione ed implementazione per l'analisi di strutture complesse* (Tesi di Dottorato di ricerca in Ingegneria Strutturale e Geotecnica, Marzo 2004, Dipartimento di Ingegneria Strutturale e Geotecnica, Università degli Studi di Genova, Tutore: Prof. Sergio Lagomarsino; Revisore: Prof. Mario Como).

Forth edition – Year 2005

Jury:

Bill Addis (Buro Happold Consulting Engineers)
Silvia Briccoli Bati (Università di Firenze)
Santiago Huerta (Universidad Politécnica de Madrid)

Winner:

Holger Eggemann, *Vereinfachte Bemessung von Verbundstützen im Hochbau. Entwicklung, historische Bemessung und Herleitung eines Nährungsverfahrens* (Dissertation an der Fakultät für Architektur der RWTH Aachen, 2003).

Fifth edition – Year 2006

Jury:

Sergio Poretti (University of Rome, Tor Vergata)
Riccardo Gulli (Alma Mater Studiorum - University of Bologna)
Joël Sakarovitch (University of Paris V)

Winner:

Hermann Schlimme, *Between Architecture, Science and Technology: the "Accademia della Vachia" in Florence, 1661-1662,* in Id. (ed. by), *Practice and Science in Early Modern Italian Building: towards an Epistemic History of Architecture*, Milano, Electa, 2006, pp. 61-96.

Sixth edition – Year 2007 *(postponed)*

Seventh edition - Year 2008

Jury:

Karl-Eugen Kurrer (Ernst & Sohn – Stahlbau, Berlin)
Patricia Radelet (Université catholique de Louvain)
Joël Sakarovitch (University of Paris V)

Ex-aequo winners:

Luc Tamboréro, *De Delhorme à de la Hire, la recherche d'une méthode universelle dans les traités de stéréotomie. Opérations géométriques et emprunts*, Mémoire présenté en vue du diplôme de l'Ecole des Hautes Etudes en Sciences Sociales, 2008, Directeur du mémoire: M. Dhombres.

Denis Zastavni, *La conception chez Robert Maillart. Morphogenèse des structures architecturales*, Thèse présentée en vue de l'obtention du grade de docteur en Sciences appliquées, Université catholique de Louvain, Faculté des Sciences appliquées, Unités d'architecture et de génie civil, 2008.

Eighth edition – Year 2009

Jury:

Stefano Bennati (Universita degli Studi di Pisa)
Rolf Gerhardt (RWTH Aachen)
Riccardo Gulli (Alma Mater Studiorum - Università di Bologna)

Ex-aequo winners:

Federica Ottoni, *La lunga vicenda delle fabbriche cupolate. Note storiche sulla stabilità, tra dibattito e sperimentazione*, Tesi di dottorato di ricerca in 'Forme e strutture dell'Architettura' (XXI ciclo) (Tutore: Prof. Carlo Blasi).

Matthew J. De Jong, *Seismic assessment strategies for masonry structures*, Submitted to the Department of Architecture on May 1, 2009 in partial fulfillment of the requirements for the degree of Doctor of Philosophy in Architecture: Building Technology (Supervisor: John Ochsendorf).

Ninth edition – Year 2010

Jury:

Patricia Radelet (Université catholique de Louvain)
Santiago Huerta (Universidad Politécnica de Madrid)
Anna Sinopoli (Università di Roma La Sapienza)

Winner:

Carlo Guastini**,** *Ponti a cassone in cemento armato precompresso. Progetto dell'attraversamento autostradale sul torrente Saultbesnon (Manche, F)*, Tesi di Laurea, Anno accademico 2008-2009, Facoltà di Ingegneria, Università di Firenze (Tutors: Prof. Paolo Spinelli, Prof. Salvatore Giacomo Morano, Prof. Vincenzo Di Naso; Co-Tutor: Dott. Daniel De Matteis).

Honorable mentions:

Frapier Christel, *Les ingénieurs-conseils dans l'architecture en France, 1945-1975: réseaux et internationalisation du savoir technique*, Thèse dirigée par le Professeur Antoine Picon, soutenue à l'Université de Paris I - Panthéon - Sorbonne le 4 Décembre 2009.

Marzia Marandola, *La costruzione in precompresso. Conoscere per recuperare il patrimonio italiano*, Manuali Ar, Il Sole 24 ore, Milano, 2009.

Tenth edition – Year 2012

Jury:

Jacques Heyman (University of Cambridge)
Santiago Huerta (Universitad Politécnica de Madrid)
Anna Sinopoli (Università di Roma La Sapienza)

Winner:

Philippe Block, *THRUST NETWORK ANALYSIS, Exploring Three-Dimensional Equilibrium*, Doctoral Philosophical Thesis in Architecture, Massachusetts Institute of Technology, June 2009.

Honorable mentions:

Alessia Bianco, *MACHINATIO. Per una storia della diagnostica architettonica precontemporanea*, Book, Aracne Editrice, Roma, September 2011.

Stefano De Santis, *Load-carrying capability and seismic assessment of masonry bridges*, Doctoral Philosophical Thesis in Civil Engineering, Università degli Studi di Roma Tre, Roma, February 2011.

Eleventh edition – Year 2015

Jury:

Stefano Bennati (Università di Pisa)
Karl-Eugen Kurrer (Ernst & Sohn – Stahlbau, Berlin)
Patricia Radelet-de Grave (Université catholique de Louvain)

Winner:

Christian Kayser, *Die Baukonstruktion gotischer Fenstermaßwerke in Mitteleuropa*, Doctoral Thesis TU München, published as a book in 2012, Editor: Michael IMHOF Verlag.

Honorable mentions:

Armande Hellebois, *Theoretical and experimental studies on early reinforced concrete structures. Contribution to the analysis of the bearing capacity of the Hennebique system*, Doctoral Thesis UL Bruxelles, published in 2013 by Presses Universitaries de Bruxelles (P.U.B.).

Roberta Fonti, *La Statica della Muratura in Pietra Grezza – Un modello interpretativo per i meccanismi fuori del piano*. Doctoral Thesis Università di Napoli Federico II, 2013.

Twelfth edition – Year 2019

Jury:

Patricia Radelet-de Grave (Université catholique de Louvain)
Santiago Huerta (Universidad Politécnica de Madrid)
Massimo Corradi (Università degli studi di Genova)

Winner:

Barbara Berger, *Der Gasbehälter als Bautypus*, PhD Thesis at the TU München, delivered in December 2017, published by the TUM University Press, München.

Honorable mention:

Gianluca Capurso, *Struttura e architettura - Indagini sul dopoguerra italiano*, draft volume currently being published by Gangemi publisher.

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Claudio Paolocci, *Presenza religiosa femminile a Genova tra XII e XVIII secolo: note di storia e di bibliografia*

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Andrea Leonardi, *Per le dimore e il collezionismo Giustiniani a Genova. Tra il cardinale Vincenzo Giustiniani olim Banca (1519-1582) e il mercante Luca Giustiniani olim Longo (1513-1583)*

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Claudio Paolocci, *Predicazione, devozioni e presenza francescana a Genova e in Liguria tra XVII e XVIII secolo*

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Fausta Franchini Guelfi, *L'iconografia delle opere di Jacopo Antonio Ponzanelli: didattica della devozione e suggestioni emotive del theatrum sacrum barocco*

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Andrea Lavaggi, *Il Cabinet d'amateur con asini iconoclasti della Società Economica di Chiavari. Iconografia, contesto storico, implicazioni religiose, significati culturali.*

Studia Ligustica 6 – 2015

Luigi Cattanei, *Scolopi liguri del primo Ottocento tra educazione, assistenza e letteratura*

Studia Ligustica 7 – 2015

Gli ex libris della Biblioteca Franzoniana di Genova. Ventisette incisori contemporanei in mostra, a cura di Andrea Lavaggi

Studia Ligustica 8 – 2015

Andrea Lavaggi, *Mirko Gualerzi pittore, disegnatore, scultore, incisore. L'Archivio generale delle sue opere*

Studia Ligustica 9 – 2017

Davide Mingozzi, *L'opera di Gaetano Isola. Un maestro di cappella nella Genova di secondo Settecento*

Studia Ligustica 10 – 2019

Filippo Comisi, *Il viaggio della prima ambasciata giapponese presso la Santa Sede (1582-1590). Percorsi e nuovi documenti dall'Archivio di Stato di Massa*

Studia Ligustica 11- 2020

Edoardo Benvenuto: un uomo del Rinascimento nel XX secolo. Dialoghi intorno alla cultura umanistica e scientifica. Atti del convegno di studi, Genova, 29 novembre 2018, a cura di Danila Aita, Giovanni Benvenuto, Massimo Corradi, Claudio Paolocci, Orietta Pedemonte

Studia Ligustica 12 – 2021

Ludovic Fina, *Le due versioni del Ragionamento di Domenico Sauli (1490-1570): le memorie e l'autodifesa di un diplomatico del Cinquecento*

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Claudio Paolocci, *La basilica di Carignano e il suo cantiere: dietro i nomi di grandi artisti. Documenti su maestranze e fornitori*

Studia Ligustica 14 – 2023

La rappresentazione della storia per la comunicazione tra immagine e testo. Dalla Genova a fumetti di Marciante all'avventura dell'originario genovese Scipione Cicala di Cuppone, a cura di Maria Linda Falcidieno

Studia Ligustica 15 – 2023

Davide Mingozzi, Adolfo Pescio: un pianista-fotografo nella Genova dell'Ottocento

Studia Ligustica 16 – 2023

Edoardo Benvenuto Prize. Collection of papers, a cura di Danila Aita, Giovanni Benvenuto, Massimo Corradi, Orietta Pedemonte