Time-dependent strength of concrete in compression and shear

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par

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Foreword

The pioneer research effort of Rüsch in the 1950s to understand the delayed failure of concrete under sustained loads motivated the scientific community to investigate the mechanics of this phenomenon and its implications in engineering practice. This topic is still subjected to an open debate, not only with respect to the detrimental effects of sustained loading on the material strength, but also with respect to its potential beneficial effects associated to larger deformation capacities and to potential stress redistributions.

In order to advance the knowledge in this topic, the different phenomena influencing the delayed response of concrete need to be considered in a coupled manner. Such approach is explored in the present thesis by Darko Tasevski, with a theoretical and experimental work. The research starts at the scale of the material, presenting an experimental investigation on the behaviour of concrete under complex loading patterns as well as a systematic comparison to existing data from the scientific literature. The aim of this part is to understand the behaviour of concrete under high levels of sustained uniaxial compression stress both in terms of deformations and strength. Based on the experimental observations and by applying a set of reasonable hypotheses, the applicability of existing creep models is extended to the nonlinear response of concrete, including secondary and tertiary creep until failure. On this basis, a general procedure to predict the response of concrete under a general loading pattern is presented and verified. In addition, an experimental campaign is also presented, at a structural scale, to the case of shear-critical reinforced concrete members without transverse reinforcement. Full-scale members were tested and data recorded by means of digital image correlation, exploring the time-dependent failure kinematics for different loading rates ranging from several seconds to several months up to failure.

The work of Mr. Tasevski resulted in a general theoretical framework allowing to address the potential strength reduction or even failure under complex sustained load histories. Beneficial effects are also considered, such as the influence of continued cement hydration and the extended deformation capacity at failure due to nonlinear creep. The theoretical framework was further extended to address realistic loading scenarios from the engineering practice, such as combinations of permanent and variable actions with different durations and intensities. On this basis, several practical design recommendations are presented including also the response of shear-critical members, and simple code-like equations for the engineering practice are developed.

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Darko Tasevski

Abstract

Sustained load actions are permanently present in concrete structures, as for example self-weight and dead loads on bridges or soil pressure on cut-and-cover tunnels. These actions may increase throughout a structure's lifetime, for instance after refurbishing or in the case of addition of new elements. It has been experimentally observed that under sustained load actions, the response of concrete may be significantly different than the response under rapid loads. Traditionally, sustained actions are treated differently at serviceability limit states (for instance delayed deflections) than at the ultimate limit state (as the potential failure under sustained load). The distinction is given by separating the time-dependent deformation due to linear creep from the strength reduction due to nonlinear creep. Although quite practical, this approach is not sufficient to understand the phenomenon of strength reduction under sustained loads and the potentially beneficial effects of the delayed response of concrete.

The present thesis introduces a comprehensive approach for the long-term analysis of concrete structures, allowing to consistently investigate the response both at serviceability and ultimate limit states. The first part of the thesis aims at establishing a theoretical framework for the evaluation of the time-dependent uniaxial compressive strength of concrete under sustained loads. This approach is accurate for any load level and loading pattern and is validated with specifically designed tests. Based on two test series on concrete cylinders under uniaxial compression with various loading rates as well as other sustained load tests from scientific literature, a failure criterion is established based on the inelastic strain capacity of concrete. For the prediction of nonlinear creep strain, the mechanical approach of Fernández Ruiz et al. (2007) is improved and extended to general loading patterns. Based on this work, it is demonstrated that the strength reduction is more severe under a constant sustained load than in the case of loads increased with a low loading rate. In addition, both the mechanical approach and the conducted tests show that the detrimental effect of sustained loading is associated with a potentially beneficial increase of deformation capacity at failure. For a detailed verification of structures with a complex loading history, the cumulative damage approach of Palmgren-Miner is adapted to account for time-dependent effects, leading to consistent results. Finally, typical engineering design situations are addressed, such as the case of application of a variable action after a period of sustained loading. The time needed for the continued cement hydration to overcome the detrimental effect of sustained loads is also investigated.

The second part of the thesis aims at verifying whether the shear strength of members without transverse reinforcement is influenced by the action of sustained loads. This topic has been little investigated in the past and current codes of practice show conflicting approaches. To contribute to this topic, two test series are conducted on slender and squat members failing in shear. Both series, as well as a comparison with other tests from the scientific literature, show that there is no marked decrease of the shear strength for longer durations of application of the load or very low loading rates compared to typical shear tests. This behaviour is supported by detailed observations performed by means of digital image correlation, allowing to quantify the contribution of the various potential shear transfer actions.

This work focuses on the long-term structural response of concrete under potentially sustained high load levels. Following the various mechanical approaches and findings, several proposals for improvement of design codes are presented and discussed.

Keywords: Sustained loading, loading rate, long-term strength, nonlinear creep, inelastic strain, deformation capacity, internal redistribution, shear strength, critical shear crack theory (CSCT), Digital Image Correlation (DIC)

Résumé

Dans les structures en béton, les actions dues à des charges soutenues sont présentes de façon permanente. C'est notamment le cas des charges du poids propre et de revêtement d'un pont, ou celui de la pression des sols exercée sur un tunnel en tranchée couverte. Ces actions peuvent augmenter au cours de la vie d'une structure, notamment lorsque cette dernière connaît une rénovation, ou lorsque de nouveaux éléments sont ajoutés. La réponse du béton sous des charges soutenues est significativement différente de celle observée pour une application rapide de la charge. Traditionnellement, les actions soutenues sont traitées différemment à l'État Limite de Service (cas des déformations différées) et à l'État Limite Ultime (cas d'une rupture potentielle sous une charge soutenue). La distinction se fait en séparant la déformation différée due au fluage linéaire, de la réduction de la résistance due au fluage non linéaire. Bien que possédant un intérêt pratique, cette approche n'est pas suffisante pour comprendre le phénomène de réduction de la résistance sous des charges soutenues ni pour approcher les potentiels effets bénéfiques liés à la réponse différée du béton.

Cette thèse introduit une approche globale pour l'analyse du comportement à long terme des structures en béton. Cette approche permet d'explorer systématiquement la réponse du béton à l'ELS et à l'ELU. La première partie de la thèse vise à établir un cadre théorique pour l'évaluation de la résistance uniaxiale à la compression du béton sous charges soutenues. La validité de cette approche est vérifiée pour tout niveau de chargement ainsi que pour des histoires de chargement complexes. Elle est également confirmée par des essais spécifiquement conçus pour cette thèse. Deux séries d'essais ont été menées sur des cylindres en béton soumis à de la compression uniaxiale sous différentes vitesses d'augmentation de la charge (certaines configurations sont tirées d'essais sous charge soutenue extraits de la littérature scientifique). Elles ont permis d'établir un critère de rupture, basé sur la capacité de déformation inélastique du béton. Pour la prédiction de la déformation non linéaire due au fluage, l'approche mécanique selon Fernández Ruiz et al. (2007) a été améliorée et étendue pour couvrir les cas de chargement généraux. À partir de ce travail, il a été démontré que la réduction de la résistance est plus importante dans le cas d'une charge constante soutenue que dans celui d'une charge progressivement augmentée. Par ailleurs, l'approche mécanique, comme les différents essais conduits, ont montré que ces effets préjudiciables sont potentiellement associés à une augmentation bénéfique de la capacité de déformation à la rupture. Pour une vérification détaillée des structures avec un historique de chargement complexe, l'approche par estimation du cumul des dommages de Palmgren-Miner est adaptée afin de tenir compte des effets liés au temps. Enfin, des situations de conception typiques de l'ingénierie sont abordées. Le cas d'une application rapide des contraintes après une période de charges soutenues est notamment traité. Le temps nécessaire à l'hydratation continue du ciment, pour surmonter les effets préjudiciables dus aux charges soutenues, est également étudié.

La seconde partie de la thèse vise à vérifier si la résistance à l'effort tranchant des éléments sans armature transversale est influencée par l'action due aux charges soutenues. Ce sujet a fait l'objet de peu de recherches dans le passé et les codes de pratique actuels montrent des approches divergentes. Pour contribuer à cette thématique, deux séries d'essais sont menées sur des éléments élancés et trapus, critiques à l'effort tranchant. Les résultats des deux séries, ainsi qu'une comparaison avec d'autres essais tirés de la littérature scientifique, montrent qu'il n'y a pas de diminution de résistance à l'effort tranchant ni pour des durées d'application de la charge augmentées ni pour des vitesses de chargement très faibles comparés aux essais de cisaillement traditionnels. Ce comportement est par ailleurs renforcé par des observations détaillées s'appuyant sur la corrélation d'images numériques. Ces dernières permettent de quantifier la contribution attachée aux différents modes de transmission de l'effort tranchant.

Ce travail se concentre sur le comportement structurel à long terme du béton soumis à des charges soutenues importantes. Sur la base des différentes approches mécaniques mobilisées et des conclusions tirées, plusieurs propositions visant l'amélioration des normes en matière de conception sont présentées.

Mots-clés : Chargement soutenu, taux de chargement, résistance à long terme, fluage non linéaire, déformation inélastique, capacité de déformation, redistribution d'effort, résistance à l'effort tranchant, théorie de la fissure critique (CSCT), corrélation d'images numériques

Zusammenfassung

Dauerlasten, wie zum Beispiel Eigengewicht und Aufbaulasten auf Brücken oder Erddruck auf Tagbautunnel, sind in Stahlbetonkonstruktionen ständig vorhanden. Sie können im Laufe der Lebensdauer eines Bauwerks zunehmen, zum Beispiel nach einer Sanierung oder im Fall von Zubauten. Experimentelle Beobachtungen zeigen, dass sich Beton unter Dauerbelastung und unter kurzweiligen Belastungen unterschiedlich verhält. Traditionell werden Dauerlasten im Grenzzustand der Gebrauchstauglichkeit (zum Beispiel verzögerte Durchbiegungen) anders behandelt als im Grenzzustand der Tragfähigkeit (als möglicher Bruch unter Dauerbelastung). Die Unterscheidung beruht darauf, dass die zeitabhängige Verformung aufgrund des linearen Kriechens von der Druckfestigkeitsabnahme aufgrund des nichtlinearen Kriechens getrennt wird. Dieser Ansatz ist zwar praktisch, reicht jedoch nicht aus, um das Phänomen der Druckfestigkeitsabnahme unter Dauerbelastung und die potenziell vorteilhaften Auswirkungen des verzögerten Deformationsvermögen von Beton zu verstehen.

Die vorliegende Arbeit stellt eine umfassende Methode für die Analyse des Langzeitverhaltens von Stahlbetonkonstruktionen vor, mit dem sowohl Grenzzustände der Gebrauchstauglichkeit als auch Grenzzustände der Tragfähigkeit konsequent untersucht werden können. Der erste Teil der Arbeit hat das Ziel, einen theoretischen Rahmen für die Bewertung der zeitabhängigen einachsigen Druckfestigkeit von Beton unter Dauerlasteinwirkung zu schaffen. Dieser Ansatz ist für alle Lastniveaus und Belastungsgeschichten anwendbar und wurde mit entsprechenden Laborversuchen validiert. Basierend auf zwei Testserien an Betonzylindern mit verschiedenen Belastungsgeschwindigkeiten sowie anderen Dauerbelastungsversuchen aus der wissenschaftlichen Literatur wird ein Versagenskriterium basierend auf dem inelastischen Verformungsvermögen von Beton festgelegt. Für die Vorhersage der nichtlinearen Kriechverformung wurde der mechanische Ansatz von Fernández Ruiz et al. (2007) verbessert und auf allgemeinen Belastungsgeschichten erweitert. Darauffolgend wird gezeigt, dass die Festigkeitsabnahme bei konstanter Dauerbelastung stärker ist als bei Lasten welche mit niedrigen Belastungsgeschwindigkeiten aufgebracht werden. Darüber hinaus zeigen sowohl der mechanische Ansatz als auch die durchgeführten Versuche, dass die nachteilige Auswirkung der Dauerbelastung mit einer potenziell vorteilhaften Erhöhung des Verformungsvermögens beim Bruch verbunden ist. Für eine detaillierte Nachrechnung von Tragwerken mit komplexer Belastungsgeschichte wird der kumulative Schadensansatz von Palmgren-Miner auf zeitabhängige Auswirkungen angepasst. Schliesslich werden typische Situationen aus der Baupraxis behandelt, z. B. die Aufbringung einer Verkehrslast nach einer Dauerbelastung. Untersucht wird auch die Zeit, welche die kontinuierliche Zementhydratation benötigt, um die nachteiligen Auswirkungen der Dauerbelastung zu überwinden.

Im zweiten Teil der Arbeit wird untersucht, ob der Querkraftwiderstand von Bauteilen ohne Schubbewehrung durch Dauerbelastungen beeinflusst wird. Dieses Thema wurde in der Vergangenheit kaum untersucht, und die aktuellen Normen beinhalten widersprüchliche Ansätze. Als Beitrag zu diesem Thema wurden zwei Testserien an schlanken bzw. gedrungenen querkraftkritischen Bauteilen mit unterschiedlichen Belastungsraten durchgeführt. Beide Testserien

sowie ein Vergleich mit anderen Tests aus der wissenschaftlichen Literatur zeigen, dass der Querkraftwiderstand bei längeren Lastdauern oder sehr niedrigen Belastungsraten mit Bezug auf typische Querkraftversuche nicht merklich abnimmt. Dieses Verhalten wird durch detaillierte Beobachtungen mittels Digitaler Bildkorrelation unterstützt, wodurch der Beitrag der verschiedenen möglichen Querkrafttragmechanismen quantifiziert werden kann.

Der Schwerpunkt dieser Arbeit liegt auf dem Langzeitverhalten von Stahlbetonkonstruktionen unter möglicherweise hohen Dauerbelastungen. Anschliessend an die verschiedenen mechanischen Ansätze und Erkenntnisse werden mehrere Vorschläge zur Verbesserung der aktuellen Normen vorgestellt und diskutiert.

Stichwörter: Dauerlasten, Dauerbelastung, Belastungsgeschwindigkeit, Dauerstandfestigkeit, Langzeitfestigkeit, nichtlineares Kriechen, inelastische Deformation, Verformungsvermögen, Spannungsumlagerung, Querkraftwiderstand, Theorie des kritischen Schubrisses (CSCT), Digitale Bildkorrelation

Riassunto

Carichi imposti nel tempo sono sempre presenti in strutture in calcestruzzo, come, per esempio, carichi dovuti al peso proprio e carichi permanenti cui sono soggetti ponti o la pressione del suolo agente su gallerie artificiali. Durante il tempo di vita di una struttura, queste azioni possono incrementare, a causa, per esempio, del rinnovo della struttura o per l'aggiunta di nuovi elementi strutturali. E' stato mostrato sperimentalmente come il comportamento del calcestruzzo soggetto a carichi imposti nel tempo può differire in maniera significativa dal caso di applicazione istantanea dei carichi. Tradizionalmente, i carichi imposti nel tempo si distinguono tra comportamento a stato limite di esercizio (incremento degli spostamenti) e a stato limite ultimo (potenziale rottura a causa di carichi imposti). Questa distinzione è data separando la deformazione nel tempo dovuta alla viscosità lineare dalla riduzione di resistenza dovuta, invece, alla viscosità non lineare. Anche se decisamente pratica, questo approccio non è sufficiente per capire il fenomeno della riduzione di resistenza a causa di un carico imposto nel tempo e i potenziali effetti benefici della risposta nel tempo del calcestruzzo.

La presente tesi introduce un approccio completo per l'analisi a lungo termine di strutture in calcestruzzo, permettendo di investigare in maniera consistente il comportamento sia a stato limite di esercizio che a stato limite ultimo. La prima parte della tesi mira ad un inquadramento teorico per la valutazione della resistenza uniassiale a compressione del calcestruzzo nel tempo sotto l'effetto di carichi imposti nel tempo. Questo approccio è accurato per ogni livello di carico e storia di carico, validato, successivamente, con test sperimentali dimensionati specificatamente. Sulla base di due serie di test su provini cilindrici in calcestruzzo soggetti a compressione uniassiale con svariate velocità di carico, come anche sulla base di altri test presenti in letteratura, un criterio di rottura è stato derivato sulla base della capacità di deformazione inelastica del calcestruzzo. Per la stima della deformazione dovuta alla viscosità non lineare, il modello meccanico proposto da Fernández Ruiz et al. (2007) è migliorato e esteso a storie di carico generalizzate. Sulla base di questo lavoro, è stato dimostrato che la riduzione di resistenza è molto accentuata nel caso di carichi imposti costanti nel tempo piuttosto che nel caso di carichi incrementati lentamente nel tempo. Inoltre, sia il modello meccanico che i test condotti, mostrano l'effetto dannoso dei carichi imposti è associato con un potenziale benefico incremento della capacità di deformazione a rottura. Per una verifica dettagliata di strutture con storie di carico complesse, l'approccio di danno cumulativo di Palmgren-Miner è stato adattato per tenere in conto degli effetti dipendenti dal tempo, conducendo a risultati consistenti. Inoltre, ci si è interessati a tipiche situazioni di dimensionamento, come il caso dell'applicazione di un'azione variabile dopo un periodo di carico imposto. Infine, è stato analizzato il tempo necessario per la continua idratazione del cemento al fine di affrontare l'effetto dannoso dei carichi imposti nel tempo.

La seconda parte della tesi mira a verificare se la resistenza a taglio di elementi senza armature a taglio è influenzata dall'azione di carichi imposti nel tempo. Questo tema è stato brevemente analizzato nel passato e le attuale normative mostrano approcci contraddittori. Per contribuire a tale tematica, due serie di test sono stati condotti, rispettivamente, su elementi snelli e elementi tozzi (rotti a taglio). Entrambe le serie, come anche il confronto con altri test estratti dalla letteratura scientifica, sembrano mostrare che non ci sia una riduzione marcata della resistenza a taglio per tempi di applicazione del carico lunghi o molto brevi, rispetto a tipici test a taglio. Questo comportamento è supportato de osservazioni dettagliate svolte con sistemi di video correlazione (DIC), permettendo di quantificare il contributo dei vari meccanismi di trasmissione del taglio.

Questo lavoro si focalizza sul comportamento strutturale a lungo temine del calcestruzzo soggetto potenzialmente ad alti livelli di carico imposto. Seguendo i vari approcci e modelli meccanici, vengono presentate e discusse varie proposte per il miglioramento delle attuali normative.

Parole chiave: Carico imposto, velocità di carico, resistenza a lungo termine, viscosità non lineare, deformazione inelastica, capacità di deformazione, ridistribuzione interna, resistenza al taglio, teoria della fessura critica (CSCT), digital image correlation

Резиме

Сопствената тежина на мостови и земјениот притисокот на тунели и потпорни ѕидови се типови на товари со постојано дејство на бетонските конструкции. Овие товари може да се зголемат во текот на животниот век на конструкцијата, на пример после санација или во случај на доградба на нови елементи. Експерименталните докази покажуваат дека при дејство на постојани товари, однесувањето на бетонот може да биде значително различно од однесувањето при краткотрајни товари. Во инженерската пракса, пристапот кон дејството на постојан товар во гранична состојба на употребливост е поинаков од пристапот во гранична состојба на лом. Разликата е во тоа што продолжените деформации во тек на време како последица на линеарно течење, се земаат во предвид одвоено од намалувањето на носивоста, како последица од оштетувањето од нелинеарно течење под висок постојан товар. Иако практичен, овој пристап не е доволен за да се разбере феноменот на намалување на носивоста при дејство на високи постојани товари, како и потенцијалните корисни ефекти кои се јавуваат од долготрајното однесување на бетонот.

Оваа докторска дисертација воведува сеопфатен приод за анализа на долготрајното однесување на бетонските конструкции, овозможувајќи доследно испитување на нивното однесување во гранична состојба на употребливост како и во гранична состојба на лом. Првиот дел од дисертацијата има за цел да утврди теоретски концепт за проценка на носивоста на бетонот на униаксијален притисок под постојани товари. Овој теоретски концепт е валиден за секое ниво на товар и за различни истории на товарење, потврдено врз база на специјално дизајнирани лабораториски експерименти. Со помош на две експериментални серии на бетонски цилиндрични проби под униаксијален притисок со различни брзини на товарење, како и експерименти со постојани товари од друга стручна литература, утврден е критериум за лом под постојан товар врз основа на капацитетот за нееластична деформација на бетонот. Механичкиот метод на Fernández Ruiz et al. (2007) за анализа на нелинеарно течење е усовршен и неговата валидност е проширена за комплексни истории на товарење. Врз основа на овој метод, се докажува дека намалувањето на носивоста на бетонот е поизразено под константен постојан товар отколку во случај на нанесување на товарот со бавна брзина на товарење. Покрај тоа, механичкиот метод и спроведените експерименти покажуваат дека негативниот ефект на постојаните товари е пропратен со потенцијално поволно зголемување на капацитетот на деформација при лом. За детална проверка на конструкции со комплексна историја на товарење, концептот на кумулативно оштетување на бетонот по Palmgren-Miner е прилагоден за поедноставено земање во предвид на штетните ефекти од постојан товар. Овој концепт покажува слични резултати со оние од механичкиот метод. Притоа се разгледуваат ситуации типични за инженерски дизајн, како што е случајот при нанесување на променлив товар по извесен период од нанесувањето на постојаниот товар (на пример проаѓање на специјален транспорт на мост). Времето потребно за продолжената хидратација на цементот да го надмини негативниот ефект од постојани товари е исто така предмет на истражување.

Вториот дел од дисертацијата истражува дали постојаните товари имаат негативно влијание на јакоста на смолкнување на бетонски елементи без арматура за смолкнување (узенгии). Оваа тема е многу слабо истражена во минатото и актуелните норми во инженерската пракса покажуваат конфликтни решенија. Со цел да се утврди евентуалното влијание, две серии на експерименти се спроведени на кратки и долги бетонски греди критични на смолкнување. Резултатите од двете серии, како и споредба со други експерименти од друга стручна литература, покажуваат дека не постои значително намалување на јакоста на смолкнување при дејство на постојани товари или многу бавни брзини на товарење во споредба со експерименти со стандардна брзина на товарење. Овој заклучок е поддржан со детални набљудувања извршени со помош на дигитална фотограметрија, овозможувајќи да се квантифицира придонесот на различните потенцијални механизми за пренесување на силата на смолкнување.

Оваа дисертација се фокусира на долготрајното однесување на бетонските конструкции под дејство на високи постојани товари. Врз база на детален механички концепт, неколку предлози за подобрување на актуелните норми се презентирани и дискутирани во детал.

Клучни зборови: Постојани товари, брзина на товарење, лом под постојан товар, нелинеарно течење, нееластична деформација, капацитет на деформација, внатрешна редистрибуција на сили, јакост на смолкнување, теорија на критична пукнатина на смолкнување (CSCT), дигитална фотограметрија (DIC)

Contents

Chapter 1 Introduction

1.1 Context and necessity of this research

1.1.1 Concrete strength under sustained loads

In the current state of knowledge, it is well established that concrete is a material sensitive to the loading rate and the duration of load application. While the rapid application of the load is shown to have a potentially beneficial influence (Das Adhikary et al. 2014; Fischer et al. 2014), high levels of sustained loads may have a detrimental influence on the compressive strength of concrete (Shank 1949; Rüsch 1960; Fouré 1985; Fernández Ruiz et al. 2007). This fact is significant in the design and assessment of concrete structures, which may experience load actions of different duration throughout their lifetime. Typical load durations can vary from accidental actions over rapid variable loads to permanent loads sustained over longer periods. A classic example of rapid actions is traffic loads on bridges (refer to *Fig. 1-1*a) whereas self-weight and soil pressure on retaining structures and cut-and-cover tunnels are considered as typical permanent actions (*Fig. 1-1*b). Hence, understanding the influence of load actions of different nature and duration on the strength of the material is instrumental for practicing engineers.

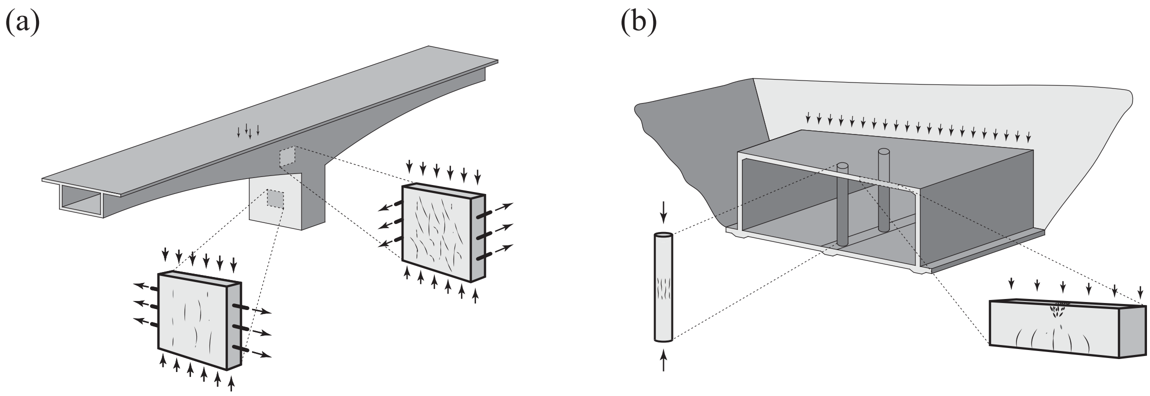

Fig. 1-1 Typical load durations on civil engineering structures: (a) live load on bridges; (b) soil pressure on cut-and-cover tunnels

The detrimental effect of sustained compressive stress larger than about 80% of its nominal strength has been already known for decades as a potential concern at the ultimate limit state (Shank 1949; Rüsch 1960). The current code provisions of different countries do not show a consensus on this effect, and most of them are based on empirical observations of simplified laboratory tests. Furthermore, there is scant knowledge of the physical mechanisms that govern these influences. On the other hand, it has been known since the early developments of concrete that its compressive strength increases with time due to the beneficial effect of the continued cement hydration (Smeaton 1791). This phenomenon may compensate for the detrimental effect of high sustained loads to a certain extent.

However, neglecting the detrimental effect of sustained loads may also be on the unsafe side. This is typically the case when the maximum design load is applied on a relatively young concrete (failure can occur before the strength increase develops). Another unsafe scenario is when an additional sustained load is applied to existing structures at a later stage, where no additional strength increase can be expected.

Moreover, loading histories in real life are composed of a combination of different actions and may lead to a rather complex alteration of the strength. The prediction of such a coupled behaviour represents a real challenge. On one hand, the increase of permanent and traffic loads throughout the structure's life may become critical for the long-term performance of bridges. On the other hand, combinations of moderate permanent actions followed by short a variable action may be less detrimental than a full permanent action. This kind of behaviour is currently not considered in codes of practice. However, taking account for the loading pattern may be of essential importance for avoiding systematic over-conservative design while at the same time fostering purposeful structural reserves.

Since the aforementioned effects apply to the design concrete compressive strength, they essentially affect all structural members which resistance directly depends on the compressive strength (compression members, compression zone due to bending, upper limit of the shear resistance in members with shear reinforcement, compression on locally loaded areas, etc., refer to *Fig. 1-1*). However, the detrimental effect of high sustained loads is accompanied with the development of nonlinear creep strain, which may potentially be beneficial by activation of stress redistributions in the structure and exploring hidden capacities.

1.1.2 Shear-critical members

In the assessment of infrastructural networks, engineers often find the need for strengthening of existing structures due to the increase of loads throughout their lifetime. It is often the shear capacity of structural members which is determining for the resistance of the structure. However, the influence of high levels of sustained loads on shear-critical members at the ultimate limit state is still poorly understood.

In the last decades, an important amount of research has been carried out to understand the influence of the nature of the load (sustained, variable) on the flexural and compressive behaviour of concrete members (Ghali et al. 2002; Gilbert and Ranzi 2011). Even though the influence of sustained and rapid actions on the compressive and tensile behaviour of concrete is acknowledged by current design codes, the influence on other potentially sensitive phenomena such as shear has not been clearly established yet. Even design codes do not show a consensus on the topic of shear strength, probably due to the scanty state of knowledge on this topic. Consequently, there is an urge for investigation of the influence of the load duration on the behaviour and resistance of concrete members in shear.

1.1.3 Open questions in current state-of-the-art

Despite all previous research efforts and advances in the knowledge, several topics with practical relevance remain still as open questions:

- The material damage under the action of high sustained loads and the continued cement hydration are two processes that occur simultaneously. The interaction of both seems to be complex due to the fact that the former one is detrimental and the latter one is beneficial for the strength development with time. Even though it is widely accepted that the continued cement hydration compensates for the effect of a high sustained load, it remains an open question if this compensation is full or only partial. In the current code provisions, there is no clear consensus on this question. Moreover, the compensation seems to be strongly dependent on the age of concrete at loading as well as on the duration of the sustained load. It needs to be investigated if the potential occurrence of a delayed failure can be described by a general failure criterion, considering the interaction of both aforementioned phenomena.

- Another open question is the strength development under different loading patterns which involve a sustain of high stress levels. Traditionally, the phenomenon of strength reduction due to sustained loads has been studied in classical creep tests with a constant sustained load level. In reality, structures are subjected to more complex loading histories, like for example gradually increased loads (typical for the construction process), slow loading rates, as well as complex loading combinations of permanent and variable actions. Hence, it needs to be clarified which types of loads have to be considered as potentially detrimental for the time-dependent development of the strength.

- The detrimental effect of high sustained loads is accompanied with additional nonlinear creep deformations, which may potentially be of a beneficial influence. The additional creep deformation may contribute to the exploitation of hidden structural reserves such as internal stress redistributions and further activation of compressive reinforcement. However, the limit between the detrimental and the beneficial effect of nonlinear creep has been poorly investigated and remains an open question. This behaviour may further depend on the geometry of the observed structural element, the presence of confinement or the compressive reinforcement ratio.

- Furthermore, the typical duration of compressive tests for the reference uniaxial strength is about 1-2 minutes whereas structural member tests have a typical duration from 20 minutes up to several hours. Even though design expressions for structural members are calibrated on the latter type of tests, they usually depend on the uniaxial compressive strength measured with the former type of tests. Up to the present, this kind of incoherence is not explicitly considered and remains an open question.

- Finally, another topic that remains poorly understood is the influence of sustained loading on the shear strength of structural concrete members without transverse reinforcement. Furthermore, most of the current code provisions do not treat this issue explicitly, probably due to the scant knowledge acquired at present. There is an urgent need on new experimental data in order to clear the ambiguity of the influence of sustained load on shear-critical members and to examine the application of the existing models for the prediction of the shear strength in sustained loading scenarios.

1.2 Objectives

From the previously identified open questions, this thesis is addressed mainly at enlarging knowledge in two fields related to material and structural response. The first one is to better understand the phenomenon of time-dependent uniaxial compressive strength of concrete under the action of high levels of sustained stress. This investigation focuses on the macroscopic behaviour of concrete and on its implications on the structural response (topics related to response at micro-structural level remain outside of the scope of this work). In the current state of knowledge, the strength reduction under different loading patterns involving high sustained stress levels is only poorly understood. The main goal of this investigation is to verify and extend the applicability of the current nonlinear creep models for prediction of the time-dependent behaviour and to establish a general failure criterion for time-dependent uniaxial compression of concrete. The investigation is based on results from scientific literature for tests under sustained constant load as well as results on slow loading rates performed in an own experimental programme. This theoretical work shall result in a verification procedure for any general loading history, which shall serve as a tool for the development of simple code-like equations and practical design recommendations.

The second objective is to investigate if the loading duration and rate have an influence on the strength of shear-critical reinforced concrete members without transverse reinforcement. The focus is given on the time-dependent development of the failure mechanism, for which observation an innovative measurement system based on digital image correlation (DIC) was established. The investigation focuses on varying loading rates up to failure, in order to investigate the contribution of different shear transfer actions and the time-dependent redistribution of stresses at different load levels. These observations shall lead to a verification of the applicability of current shear design models (with emphasis on the Critical Shear Crack Theory) as well as practical design recommendations for validation or improvement of current codes of practice.

1.3 Thesis structure

The present thesis is organized in 6 chapters:

- Chapter 1 introduces the topics studied in the present thesis, followed by the main objectives and scientific contributions as well as a list of publications.

- Chapter 2 presents the state of the art on the time-dependent behaviour of concrete both in terms of deformation and development of concrete properties with time. Furthermore, it gives an overview of the current knowledge on the time-dependent strength of structural concrete members. The main aim of this chapter is to give a context to the main questions addressed in this thesis.

- Chapter 3 presents a theoretical framework developed for consistent investigation of the time-dependent uniaxial response of concrete under high sustained loads. The framework is focused on the prediction of the development of nonlinear creep strains potentially leading to failure, as well as on the establishment of a failure criterion for concrete under high levels of sustained stress. It is based on an experimental campaign of normal strength concrete under varying strain and stress rates, which is also presented in this chapter. Finally, a validation of the framework with further experimental evidence from scientific literature is presented.

- Chapter 4 presents the extension of the applicability of the theoretical framework for any complex loading pattern involving high levels of sustained stress. The main aim of this chapter is to develop engineering design rules in form of simple code-like equations and to propose practical design recommendations. For this aim, the extended theoretical framework is explored for addressing typical design situations from the engineering practice, as for instance the time duration needed for the continued cement hydration to overcome the detrimental effect of sustained loads. Furthermore, the particular case of application of a variable action after a period of permanent load action is studied.

- Chapter 5 presents a detailed investigation of the influence of the loading rate on the strength of shear-critical reinforced concrete members without transverse reinforcement. The results of an experimental campaign on slender and squat beams are presented, with loading rates ranging from several seconds to several months until failure. The development of the failure mechanism is discussed and a detailed investigation is performed on the observed development of crack patterns with time. Furthermore, the contribution of the different shear transfer actions is evaluated. The observations lead to a validation of an existing model for shear strength prediction (CSCT) and help to draw several practical design recommendations.

- Chapter 6 presents the conclusions of the previous chapters and gives an outlook for possible future research within and beyond the scope of this thesis.

It is worth noting that Chapters 3 to 5 are pre- and postprint versions of publications submitted or published in scientific journals. However, the numbering and the layout of the artwork have been adapted to match the layout of the present document. In addition, every chapter of the thesis contains its own section of Notation, References, and Appendix (if applicable), in order to conform with the rules for a paper-based thesis.

Finally, five appendices presented at the end of the document complete the previous chapters with additional information.

1.4 Scientific contributions of the thesis

The main scientific contributions on the uniaxial compressive behaviour of concrete under the influence of different load durations and rates are:

- Carrying out two unique experimental series on concrete cylinders under uniaxial compression with varying strain and stress rates at high stress levels. A detailed analysis of their results together with many other tests available in the scientific literature has helped to analyse the portions of instantaneous strain, linear creep strain, nonlinear creep strain and shrinkage strain, as well as to test the applicability of the monotonic response of concrete as an indicator of the inelastic strain capacity.

- Improvement of the existing analytical tools for nonlinear creep strain prediction and validation of the affinity hypothesis for nonlinear creep. This improvement has helped to establish a general failure criterion for time-dependent uniaxial compression of concrete under high sustained stress.

- Extending of the analytical framework for the prediction of nonlinear creep strain development and delayed failure under complex loading histories. For this purpose, the cumulative damage estimation approach of Palmgren-Miner has been adapted.

- Performing a series of parametric studies for the practical design case of application of a variable action after a period of sustained load (including the influence of the duration of the variable action).

- Proposal for improvement of the existing code provisions for time-dependent uniaxial compressive strength of concrete under high sustained loads and practical design recommendations.

The main scientific contributions on the behaviour of shear-critical reinforced concrete members without transverse reinforcement under the influence of different load durations and rates are:

- Carrying out two unique experimental series on shear-critical slender and squat members without transverse reinforcement subjected to varying loading rates up to failure. Detailed measurements were performed on the entire deformation field by means of digital image correlation (DIC).

- Detailed investigation of the time-dependent development of the failure mechanism and the crack pattern, as well as the contribution of different shear transfer actions and redistribution of stresses.

- Validation of existing models for shear strength prediction based on the experimental observations (with emphasis on the Critical Shear Crack Theory).

- Proposal for improvement of the existing code provisions for the resistance of shear-critical reinforced concrete members without transverse reinforcement under the influence of sustained load and practical design recommendations.

1.5 List of publications

Tasevski D, Fernández Ruiz M and Muttoni A (2019), "Influence of sustained actions and loading rate on the strength of reinforced concrete structures.", FEDRO Report, *submitted for publication*.

Tasevski D, Fernández Ruiz M and Muttoni A (2019), "Influence of load duration on shear strength of reinforced concrete members.", Scientific article, *submitted for publication*.

Tasevski D, Fernández Ruiz M and Muttoni A (2019), "Assessing the compressive strength of concrete under sustained actions: from refined models to simple design expressions." *Structural Concrete*, 2019, 1-15.

Tasevski D, Fernández Ruiz M and Muttoni A (2018), "Compressive strength and deformation capacity of concrete under sustained loading and low stress rates." *Journal of Advanced Concrete Technology*, 16(8), 396-415.

Tasevski D, Fernández Ruiz M and Muttoni A (2016), "Behaviour of concrete in compression and shear under varying strain rates: from rapid to long-term actions", in Proceedings of the 11th fib International PhD Symposium in Civil Engineering, Tokyo, 29-31 August 2016, pp. 881–886.

Tasevski D, Fernández Ruiz M and Muttoni A (2015), "Analogy between sustained loading and strain rate effects on the nonlinear creep response of concrete", in CONCREEP 10, International Conference on Mechanics and Physics of Creep, Shrinkage and Durability of Concrete and Concrete Structures, Vienna, 21-23 September 2015, pp. 1187-1193.

1.6 References

Das Adhikary S, Li B, and Fujikake K (2014), "Effects of high loading rate on reinforced concrete beams", *ACI Structural Journal*, vol. 111, no. 3, pp. 651–660.

Fernández Ruiz M, Muttoni A, and Gambarova PG (2007), "Relationship between nonlinear creep and cracking of concrete under uniaxial compression", *Journal of Advanced Concrete Technology*, vol. 5, no. 3, pp. 383–393.

Fischer I, Pichler B, Lach E, Terner C, Barraud E, and Britz F (2014), "Compressive strength of cement paste as a function of loading rate: Experiments and engineering mechanics analysis", *Cement and Concrete Research*, vol. 58, pp. 186–200.

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Rüsch H, (1960), "Researches toward a general flexural theory for structural concrete", *ACI Journal*, vol. 57, no. 1, pp. 1–28.

Shank JR (1949), "Plastic flow of concrete at high overload", *ACI Journal*, vol. 20, no. 6, pp. 493–498.

Smeaton J (1791), "A narrative of the building and a description of construction of the Eddystone lighthouse with stone", Publisher: H. Hughs, London, 198 p.

Chapter 2 State of the art

This chapter presents the state of the art on the time-dependent behaviour of concrete both in terms of deformation and development of concrete properties with time. Furthermore, it gives an overview of the current knowledge on the time-dependent strength of structural concrete members. This work has not been published as an independent manuscript, and its aim is to provide a common perspective to the different state-of-the-art sections included in chapters 3-5.

2.1 Time-dependent deformations of concrete

The time-dependent deformations of concrete subjected to a sustained load are characterized through two main phenomena, the first being shrinkage and the latter being creep. As a result of these phenomena, delayed strains develop in concrete. By definition, shrinkage strains are independent of the state of stresses in concrete so they represent a pure viscous behaviour of the material. On the contrary, creep is dependent on both the magnitude and the duration of concrete stresses. In this thesis, the emphasis is put on the phenomenon of creep, which has traditionally been regarded as a concern at the serviceability limit state.

An observation that the properties of concrete vary with time and depend on its age were early acknowledged since the first modern developments of concrete (Smeaton 1791). The structural implications of this behaviour were also soon detected, as for instance the observation of excessive deflections with time in the Le Veurdre bridge by Freyssinet in 1910 (Fernández-Ordóñez 2018). Today creep is still a hot topic in the research domain of concrete. A considerable number of books on this phenomenon can be found, treating different aspects and studying various parameters that govern the phenomenon. Some notable works can be consulted here (Neville 1955, 1966; Setzer and Wittmann 1982; Bažant 1988; Ghali et al. 2002; Gilbert and Ranzi 2011). Most of these investigations focus on the elastic domain of response of concrete, where concrete stresses are normally assumed to amount not more than approximately 40% of the short-term concrete strength. It has been well established that in this stress domain, the creep strain can be linearly related to the instantaneous elastic strain. Hence, the creep strain is traditionally estimated by a linear creep coefficient in the following manner:

$$\varepsilon_c(t, t_0) = (1 + \varphi_{lin}) \cdot \varepsilon_{c0} \tag{2.1}$$

Several models exist for prediction of the linear creep coefficient, notably the *fib* MC2010 (fib 2013b) and the B4 Model of Bažant (Bažant 2015). These models take into account the influence of different parameters which may affect the creep behaviour, such as temperature, relative humidity, curing conditions, the geometry of the structural member etc. However, for load levels higher than 40% of the short-term strength, the linear relation of creep strain to the elastic strain is lost, and linear creep models aren't able to reproduce this behaviour. In the current state of the art, there exist few expressions that propose to correct the linear creep coefficient for the nonlinear effect in the following manner:

$$\varepsilon_c(t, t_0) = (1 + \eta \cdot \varphi_{lin}) \cdot \varepsilon_{c0} \tag{2.2}$$

where η is usually valid for sustained loads up to $\sigma_c/f_c = 0.6$ and for creep strains developed after a long time of sustained load action. The *fib* MC2010 (fib 2013b) proposes an empirical expression for the coefficient η (Eq. (5.1-74a) in *fib* MC2010) which reads:

$$\eta\left(\frac{\sigma_c}{f_{cm}(t_0)}\right) = e^{1.5\cdot(k_\sigma - 0.4)} \quad \text{for } 0.4 < k_\sigma < 0.6 \tag{2.3}$$

where $k_\sigma = \sigma_c / f_{cm}(t_0)$. This function represents a non-smooth shape, as it is zero for values of $\sigma_c/f_c < 0.4$ and it has a kink at the value $\sigma_c/f_c = 0.4$. An improved expression (also empirical) with a smooth shape and an extended domain of validity (up to $\sigma_c/f_c = 0.7$) was proposed by (Fernández Ruiz et al. 2007):

$$\eta\left(\frac{\sigma_c}{f_{cm}(t_0)}\right) = 1 + 2 \cdot \left(\frac{\sigma_c}{f_{cm}(t_0)}\right)^4 \quad \text{for } 0.4 < \frac{\sigma_c}{f_{cm}(t_0)} < 0.7 \tag{2.4}$$

This expression has been validated with additional test results from scientific literature, as shown in *Fig. 2-1*, and seems to give a satisfactory prediction even for stress levels up to $\sigma_c/f_c = 0.8$.

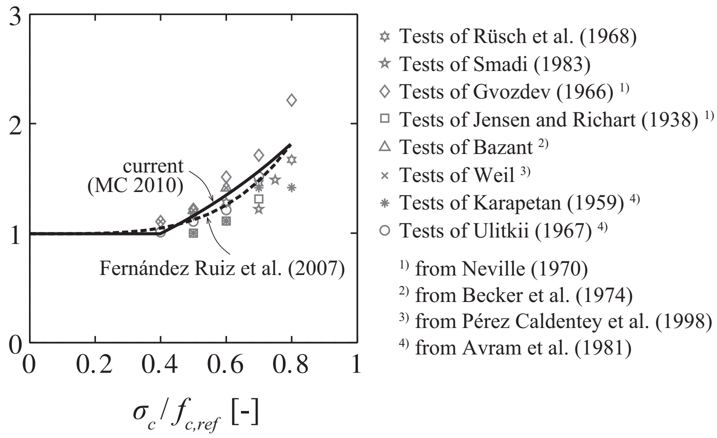

Fig. 2-1 Comparison of the fib MC 2010 expression for nonlinear creep and the one proposed by (Fernández Ruiz et al. 2007) and used in the analytical scheme of this chapter

The validity of the current nonlinear creep expressions for the prediction of creep strains developed for shorter durations of sustained load remains an open question and will be addressed in Chapter 3. Another open question is the validity of this expressions for load levels higher than $\sigma_c/f_c = 0.8$, which will also be addressed in Chapter 3.

2.2 Development of concrete properties with time

Concrete properties such as the strength and modulus of elasticity are known to evolve with time. The development of these properties in a structural member subjected to sustained load is complex, normally due to several phenomena that occur at the same time:

- The concrete strength and modulus of elasticity are known to increase with time due to continued cement hydration. This increase is significant in the first several months or years after casting (depending on the cement hardening properties and curing conditions), and the strength value stabilizes afterward;

- The concrete strength increase due to continued cement hydration may be magnified under the action of moderate levels of sustained stress;

- The concrete strength may reduce under the action of high levels of sustained stress. This is due to the development of nonlinear creep strains, which are associated with material damage (propagation and growth of microcracks) and reduce the strength with respect to short-term loading.

The following sections discuss the listed phenomena separately.

2.2.1 Influence of continued cement hydration

The hydration of cement is a continuous reaction which can take place over many years after casting. It is well known that thanks to the continued cement hydration the concrete strength increases with time, as early observed by Smeaton (Smeaton 1791). Over the past decades, the combined effects of time, curing temperature and humidity have been thoroughly investigated (Saul 1951; Rastrup 1954; Freiesleben Hansen and Pedersen 1977). Current codes of practice describe the development of concrete compressive strength with time by means of a continuous empirical function taking into account for the strength class of cement (*fib* MC2010) and curing conditions (ACI Committee 209 (ACI 2008)). The *fib* MC2010 (fib 2013b) proposes the following empirical expression (Eq. (5.1-50) of *fib* MC2010):

$$f_{cm}(t) = \beta_{cc}(t) \cdot f_{cm,28} \tag{2.5}$$

with

$$\beta_{cc}(t) = e^{s \cdot \left(1 - \sqrt{\frac{28}{t}}\right)} \tag{2.6}$$

where the time *t* is measured in days and *s* is a coefficient which depends on the cement strength class and takes values from *s* = 0.2 for rapid to *s* = 0.38 for slow hardening cement. The evolution of the concrete modulus of elasticity with time is described in *fib* MC2010 in a similar manner (Eq. (5.1-56) of *fib* MC2010):

$$E_{ci}(t) = [\beta_{cc}(t)]^{0.5} \cdot E_{ci,28} \tag{2.7}$$

Fig. 2-2 plots the evolution of the normalized compressive strength and modulus of elasticity with time according to *fib* MC2010:

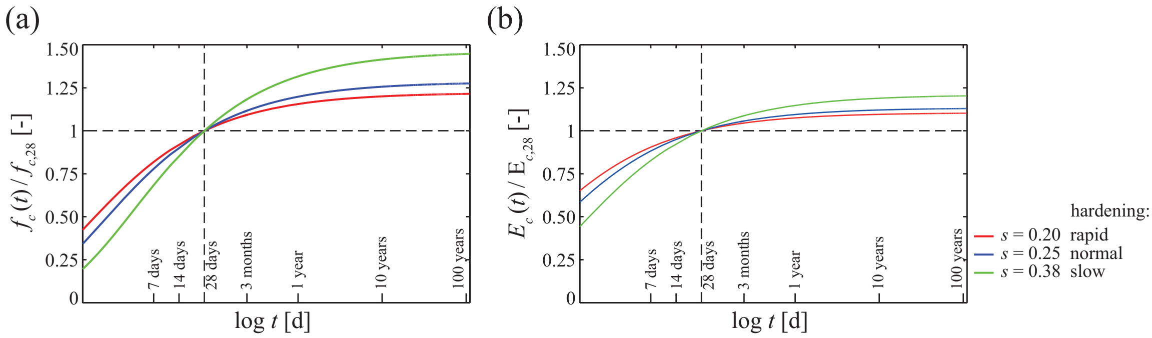

Fig. 2-2 Development of: (a) concrete compressive strength; and (b) modulus of elasticity with time, according to *fib* MC2010

The strength development with time in a structural member is more complex and may depend on other parameters such as member geometry, position in the structure and curing conditions. Furthermore, it may strongly depend on the loading history which the concerned structural member is subjected to.

The contribution of this phenomenon to the concrete compressive strength development under the action of sustained loads is still an open question. As shown in the following two sections, the magnitude of the sustained load plays an important role too. In the scope of this thesis, the combined effect of the continued cement hydration together with the action of high sustained loads will be investigated in detail.

2.2.2 Concrete strength under moderate sustained load

Several studies have shown that the strength increase of concrete with time due to continued cement hydration may be magnified under the action of a moderate sustained load (Washa et al. (1950), Freudental et al. (1958), Rodriguez (1960), Stöckl (1960), Shah et al. (1970), Hughes et al. (1970), Coutinho (1977), Cook et al. (1980), Abdel-Jawad et al. (1992)). This has been manifested by reloading of test specimens to failure after the action of a moderate sustained load. The results of several authors have been plotted in *Table 2-1*, which shows that the average magnification of the strength increase of all studies is approximately 5 %.

The potentially favourable effect of moderate sustained loads seems to be strongly dependent on the sustained load level. However, this phenomenon is still very unclear and the conclusions are based on scanty experimental evidence. It is worth noting that higher load levels may have a detrimental effect on the strength, as discussed in the following section. Furthermore, since the load level is usually normalized to the strength at the age at load application, for loads applied at an early concrete age the internal concrete stress level decreases rapidly as the strength increases with continued cement hydration.

After a careful study of the available scientific literature, it can be concluded that the favourable effect of a moderate sustained load is stronger if the load is applied at early concrete ages (from one day on). Some studies show that a sustained load applied at an early concrete age for only couple of hours (Shah and Chandra 1970; Coutinho 1977) or even a simple instantaneous preloading before reloading to failure (Claisse and Dean 2013) may result with increase of the strength. However, if the sustained load is applied on an old concrete, the effect of additional strength increase seems to be negligible.

The phenomenon of additional strength increase due to the action of moderate sustained loads is not in the scope of this thesis.

Table 2-1 Experimental programmes performed in the scientific literature for the influence of a moderate sustained load on the compressive strength of concrete

Reference	$f_{c,28}$ [MPa]	t_0 [d]	Δt [d]	σ_c/f_c [-]	$f_c(t)/f_{c,28}$ [-]	$f_c(t)/f_c(t_0)$ [-]
Washa et al., 1950	10 - 45	28	3835	0.18 - 0.25	1.39	0.95
					1.20	1.06
Freudenthal et al., 1958	30 - 60	28	168 - 287	0.17 - 0.63	1.26	1.02
					1.03	1.04
Rodriguez, 1960	-	28	500	0.10 - 0.20	-	1.10
Stöckl, 1960	17 - 20	3	28	0.50 - 0.90	2.21	1.04
			227		2.56	1.12
	9 - 50	28	28		1.12	1.04
			190 - 260		1.25	1.07
Shah et al., 1970	28	28	0.146	0.60 - 0.70	-	1.06
Hughes et al., 1970	21 - 28	3	25	0.50	-	1.08
		150				1.05
Coutinho, 1977	19.2	7	3	0.21 - 0.89	-	1.05
		7	0.125 - 28	0.81		1.07
		90	0.333 - 28	0.81		1.05
		3 - 120	3 - 28	0.77 - 0.90		1.11
Cook et al., 1980	27 - 42	28	30	0.30 - 0.60	1.07	1.01
Abdel-Jawad et al., 1992	21 - 34	3	4 - 87	0.60 - 0.90	-	1.01
					AVERAGE	**1.05**

2.2.3 Concrete strength under high sustained load

In the middle of the last century, Shank (Shank 1949) and Rüsch (Rüsch 1960) have discovered a detrimental effect of sustained compressive stresses larger than about 75% of the nominal concrete compressive strength. This phenomenon, related to a tertiary creep behaviour, may potentially be a concern for structures at the ultimate limit state. It is believed that the detrimental effect of high sustained stresses was the reason of collapse of the Zumrut apartment building in Turkey in 2004 (Kaltakci et al. 2007), where evidence of heavy damage due to creep could be observed in the basement floor columns (refer to *Fig. 2-3*).

Fig. 2-3 Zumrut apartment building in Turkey: (a) Structural system, (b) heavy damage in the basement columns due to nonlinear creep, and (c) Collapse aftermath (taken form (Kaltakci et al. 2007))

The comprehensive research work of Rüsch (Rüsch 1956, 1960; Rüsch et al. 1968) resulted with the establishment of a failure limit for concrete under sustained load, governing for stress levels above the so-called "sustained load strength" (critical degree of loading above which delayed failure is possible, found to be at $\sigma_c/f_c \approx 0.75$, refer to *Fig. 2-4*a). For sustained loads below the sustained load strength, concrete would reach the so-called creep limit (strains increase with time by creep deformations but not leading to failure, *Fig. 2-4*a). This pioneer work was followed by several researchers, some of which have studied the failure under compressive creep of normal strength concrete (Shah and Chandra 1970; Awad and Hilsdorf 1971; Diaz and Hilsdorf 1971; Stöckl 1972; Wittmann and Zaitsev 1974; Coutinho 1977; Smadi et al. 1985; Fouré 1985; Nechvatal et al. 1994; El-Kashif and Maekawa 2004; Maekawa and El-Kashif 2004; Fernández Ruiz et al. 2007; Suryanto et al. 2013; Wang et al. 2015; Schlappal et al. 2017) as well as of high strength concrete (Ngab et al. 1981; Smadi 1983; Smadi et al. 1985; Han and Walraven 1994; Müller et al. 2010; Anders 2012).

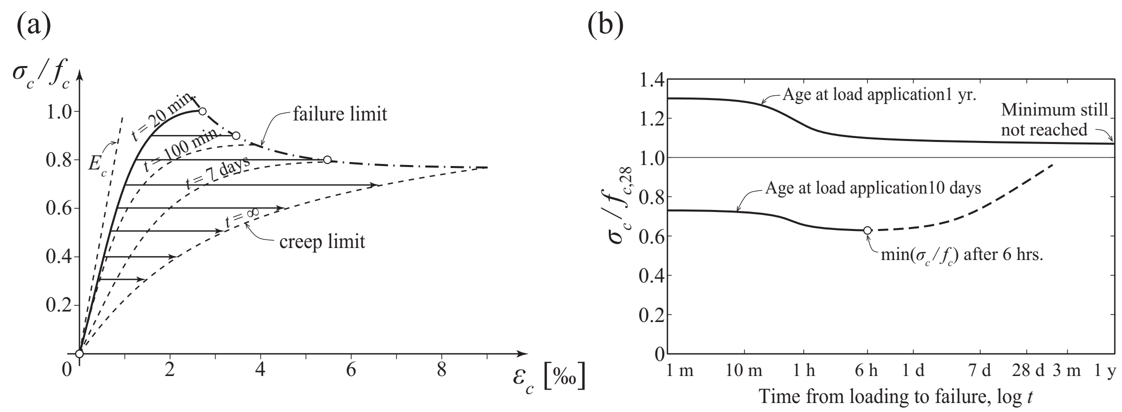

Fig. 2-4 Sustained load strength definition by Rüsch (adapted from (Rüsch 1960)): (a) Influence of load intensity and duration on concrete strain; (b) Influence of age at loading on strength under sustained load

Depending on the age at load application, the strength increase due to continued cement hydration may take place in the same time with the material degradation under high sustained loads and may or may not compensate for the detrimental effects of sustained loads. For instance, in the design of new structures where a design value based on the 28-day strength is used, a partial or even full compensation may be expected. This phenomenon was recognized by Rüsch (Rüsch 1960) who found that there exists a minimum of the counteraction of both effects, which strongly depends on the age of load application t_0 (*Fig. 2-4*b). However, there are several cases where the sustained loading effect is not compensated by the continued cement hydrations: (i) when a high sustained load is applied on a relatively young concrete (failure can occur before the strength increase due to continued cement hydration can develop), (ii) when higher load levels are applied in a later stage of life of the structure (particularly the case of rehabilitation of existing structures, where additional dead loads are applied but no additional strength increase due to continued cement hydration can be expected, as concluded in *Fig. 2-4*b).

In the current state of the art, the concrete compressive strength in case of a constant sustained loading applied at time t_0 and with a duration of t-t_0 can be estimated by (Eq. (5.1-53) of *fib* MC2010 (fib 2013b):

$$f_{cm}(t) = f_{cm,28} \cdot \beta_{cc}(t) \cdot \beta_{c,sus}(t, t_0) \tag{2.8}$$

with

$$\beta_{c,sus}(t, t_0) = 0.96 - 0.12 \cdot \left(\ln\left(72 \cdot (t - t_0)\right)\right)^{1/4} \tag{2.9}$$

where the time t and the age at load application t_0 are measured in days. *Fig. 2-5* plots the normalized concrete compressive strength evolution with time under high sustained load according to *fib* MC2010:

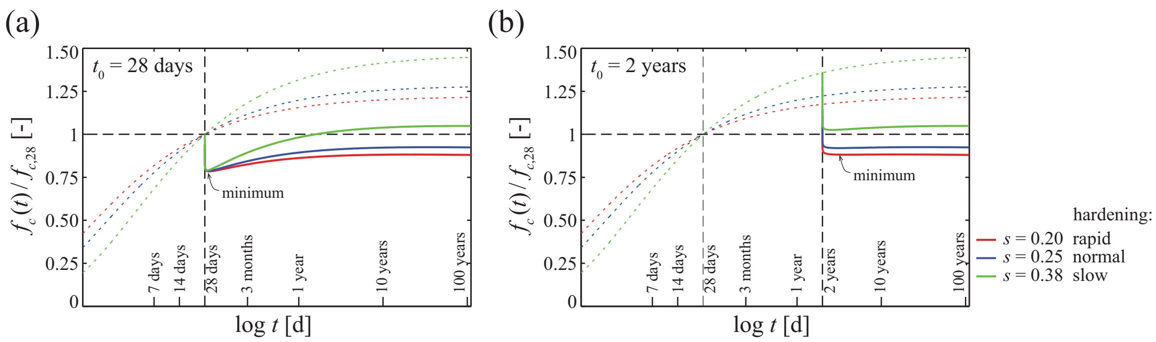

Fig. 2-5 Development of concrete compressive strength under high sustained load according to fib MC2010 for: (a) t_0 = 28 days; and (b) t_0 = 2 years

As it is shown in *Fig. 2-5*, the counteraction of both phenomena described by Rüsch (*Fig. 2-4*b) can be described by independent empirical functions. However, at present, there is no physical description of this complex behaviour and the empirical phenomena have been based on a limited number of tests (fib 2013a). One of the main objectives of the present work is to better understand the interaction of both phenomena as well as their dependence on the concrete age.

In several codes of practice (for instance *fib* MC2010 (fib 2013b), SIA 262:2013 (SIA 2013) and Eurocode 2:2004 (CEN 2004) in some countries), the effect of a high sustained load is considered by a reduction of the design concrete compressive strength by a factor varying between 0.85 and 1.0. The value of 1.0 is typically used in the design of new structures, for which the governing design situation occurs when the effect of continued cement hydration is sufficient to compensate the strength decrease due to sustained loading. The value of 1.0 is also used if the compressive stress at design level due to sustained loading does not exceed 85 to 90% of the compressive strength. The lower value of 0.85 refers to the more severe condition when the concrete strength is assessed after some years/decades and the compressive stress at design level due to sustained loading exceed 85 to 90% of the compressive strength. Since this reduction is applied to the design concrete compressive strength, it affects all structural members which resistance directly depends on the compressive strength (compression members, compression zone due to bending, the upper limit of the shear resistance in members with shear reinforcement, compression on locally loaded areas, etc.).

However, in the Eurocode 2:2004 the reduction factor is a Nationally Determined Parameter, and there is no clear consensus between the countries about the value that should be used (Muttoni et al. 2017). Some countries use the recommended value 1.0, some use the conservative value 0.85, some use the value 0.85 only for normal force and bending and the other distinguish between sustained loading (0.85) and short-term loading (1.0) as described in the previous paragraph. Furthermore, the American code ACI 318-14 (ACI 2014) takes implicitly into account for the effect of high sustained load in highly loaded columns (also used as a justification for minimum reinforcement in compression members), as well as for the strength reduction in concrete struts. Hence, the different codes of practice do not show a consensus on this topic.

The mechanical origin of the behaviour of concrete under the action of high sustained stresses as well as its interaction with the beneficial effect of continued cement hydration remains until today an open question. The present thesis aims to better understand this behaviour in order to develop a mechanically based model for prediction of the time-dependent concrete compressive strength. It should serve as a basis for validation of existing codes of practice and for providing further practical design recommendations.

2.3 Uniaxial compressive strength vs. structural member strength

The characteristic value of the compressive strength of concrete is typically determined in a test of duration of about 1-2 minutes (according for instance to ISO 1920-4:2005 (ISO 2005), either on fabricated cylindrical and cube specimens in the case of new structures or on core samples extracted from existing structures (refer to *Fig. 2-6*a). The design value of the compressive strength, usually used in structural member design at ultimate limit state, is obtained by a further decrease by a partial safety factor accounting for uncertainties related to the material nature. On the contrary, laboratory tests on structural members (*Fig. 2-6*b) such as columns in compression or beams and slabs in bending, shear and punching have typical duration from 20 minutes up to several hours. Design formulae for structural members in code provisions are calibrated on these tests; nevertheless, most of them depend directly on the design value of concrete compressive strength. As a consequence, one could conclude that their prediction is on the rather conservative side due to the implicit account of a portion of strength reduction due to the lower rate of loading (associated with longer load sustain).

In the current state of the art this kind of hidden reserve has not been studied explicitly, and it remains an open question which will be addressed in the present thesis.

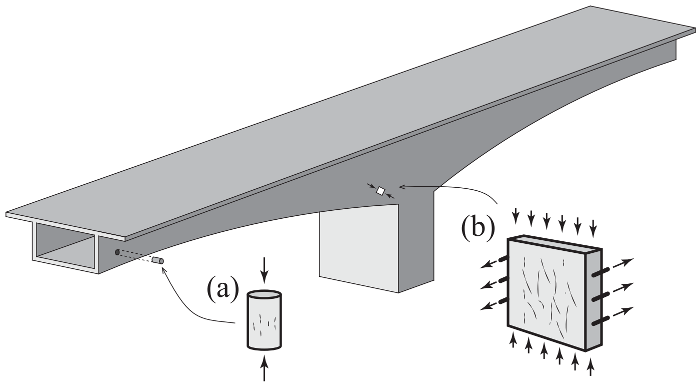

Fig. 2-6 Difference between (a) uniaxial strength and (b) structural member strength

2.4 Time-dependent strength of structural concrete members

The time-dependent behaviour of reinforced concrete members has been a major subject of research in the last decades. Several notable studies can be found on the time-dependent deflections of concrete members in bending at the serviceability limit state (Bakoss et al. 1982; Nie and Cai 2000; Ghali et al. 2002; Gilbert and Ranzi 2011; Reybrouck et al. 2017). The long-term deflections and the shift of the neutral axis can be well predicted by means of simplified cross-sectional analysis with the application of the age adjusted effective modulus approach (Ghali et al. 2002). On the other hand, the recent development in material models for time-dependent strength prediction has allowed their application to study the structural response of concrete members at the ultimate limit state. Hamed (Hamed 2012) has applied some of those models (including the model of Fernández Ruiz et al. (Fernández Ruiz et al. 2007) presented in section 2.1) to an analytical scheme for prediction of the time-dependent nonlinear behaviour of beams in bending, producing interesting findings that creep may continuously decrease the bending strength, especially at the presence of an additional axial compressive force.

However, very few studies exist on the shear behaviour of concrete members under sustained load. A notable finding has been introduced by Nie and Cai (Nie and Cai 2000) for members with transverse reinforcement, showing that for specimens with developed diagonal cracking (for a shear-span-to-effective-depth-ratio $a/d = 2.5$), shear effects contribute from 13 – 35 % of the total deflection due to sustained load and shall not be neglected in the calculation. In a numerical study based on those results, Ferreira et al. (Ferreira et al. 2016) have found that time-dependant strains may lead to important damage and reduction of the shear capacity of shear critical members with restrained boundary conditions. The shear strength reduction increases for increasing axial restraint force and the reduction was found to be associated with the loss of the concrete contribution in the shear-carrying mechanism (and not the transverse reinforcement).

The influence of sustained loads on the shear resistance of concrete members without shear reinforcement is however a very young research topic. There exist only a limited number of findings in this domain. The first experimental evidence on shear-critical members without transverse reinforcement under sustained loading has been produced by Sarkhosh (Sarkhosh 2013, 2014). His work was dedicated to the investigation of the time-dependent crack growth in slender beams with a shear-span-to-effective-depth-ratio $a/d = 3.0$, subjected to elevated sustained loads of 87% - 95% of the average short-term shear capacity. A clear evidence of potential delayed shear failure was presented by two beams failing under sustained loading (one after 2.5 hours and one after 44 hours). However, the majority of the beams did not fail under sustained load and needed to be reloaded to attain failure. A detailed observation of the time-dependent crack development in Sarkhosh's experiments suggests a stable crack length propagation in the first days or weeks after the sustained load application, with a convergent behaviour afterwards. As a general conclusion of this work, no significant decrease of the shear strength due to sustained loads could be observed.

Another notable experimental study on slender beams ($a/d = 3.0$) has been performed by Saifullah et al. (Saifullah et al. 2017) by applying loading rates 100 and 1000 times slower than the reference loading rate (which was about an hour to failure). A general increase of midspan deflection, crack opening and compressive strain at the top fiber with time could be observed, as well as an overall increase in the ultimate shear capacity (yet the results are somewhat scattered). On the other hand, the diagonal cracking has been observed to occur at the approximately same load for all loading rates. It is worth noting that some of the tests were performed with load control and some with displacement control. A numerical study by the same authors (Saifullah et al. 2018) concludes the rather opposite effect, namely that the ultimate shear capacity shall decrease for lower loading rates. This study suggest that the strength increase may possibly be related to the increase of fracture energy for low loading rates (shown by Wittmann et al. (Wittmann et al. 1987)).

In a study based on the long-term deformation monitoring of underground box culverts, Maekawa et al. (Maekawa et al. 2016) have identified the possible delayed shear failure of concrete members under a sustained soil pressure. The geometrical conditions of the culverts suggest a rather squat-beam-like behaviour (diagonal shear failure and direct strut action expected). Diagonal cracking in the slab near the rigid corner of the concrete box was detected with excessive deflections (10 times higher than the design deflection), and boreholes have confirmed the existence of an inclined shear crack. In a combination with a numerical study, the authors concluded that the delayed shear cracking have occurred and propagated under a sustained load lower than the maximum shear capacity. In a further numerical study on deep beams ($a/d = 1.0 - 2.4$), Bugalia and Maekawa (Bugalia and Maekawa 2017) show that arching action ad direct strut action are strongly influenced by sustained compression loads, suggesting a decrease of strength for sustained loading.

In conclusion, the influence of sustained loading on the shear strength of structural concrete members without transverse reinforcement is very ambiguous and remains a topic of further research. It is in the scope of the present thesis to clear this ambiguity. Furthermore, current code provisions do not show a consensus on how to address this phenomenon, and this work aims to provide some practical design recommendations on this topic.

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Chapter 3 Compressive strength and deformation capacity of concrete under sustained loading and low stress rates

This chapter presents a theoretical framework developed for consistent investigation of the time-dependent uniaxial response of concrete under high sustained loads. The framework is focused on the prediction of the development of nonlinear creep strains potentially leading to failure, as well as on the establishment of a failure criterion for concrete under high levels of sustained stress. It is based on an experimental campaign of normal strength concrete under varying strain and stress rates, which is also presented in this chapter. Finally, a validation of the framework with further experimental evidence from scientific literature is presented.

The chapter is a postprint version of a publication in a peer-reviewed academic journal. The authors are Darko Tasevski (PhD Candidate), Miguel Fernández Ruiz (Senior lecturer and thesis director) and Aurelio Muttoni (Professor and thesis director), and the full reference can be found here:

> **Tasevski, D., Fernández Ruiz, M. and Muttoni, A., (2018),** "Compressive strength and deformation capacity of concrete under sustained loading and low stress rates." Journal of Advanced Concrete Technology, 16(8), 396-415 (https://www.jstage.jst.go.jp/article/jact/16/8/16_396/_article/-char/en)

The main contributions of Darko Tasevski are the following:

- Preparing and performing the full experimental campaign
- Detailed analysis of the test results
- Development of the analytical approach in collaboration with the thesis supervisors
- Numerical implementation of the analytical approach
- Performing the parametric analyses
- Production of the artwork in the article
- Preparation of the manuscript of the article

The second and third author (thesis directors) contributed to the understanding of the studied phenomena, interpreting of the experimental and analytical findings and writing of the final manuscript.

3.1 Abstract

This paper investigates the behaviour of concrete failing under high stress levels and subjected to different types of loading. The aim of this investigation is to clarify the development of linear and nonlinear creep strains and how they relate to material damage and eventual failure. This research is supported on the results of a new experimental programme performed on concrete cylinders tested in uniaxial compression under varying strain and stress rates. The results of this programme allow investigating the influence of the loading history on the material response in terms both of its strength and deformation capacity. On this basis, a failure criterion related to the inelastic strain capacity of concrete is defined. Such failure criterion, showing consistent agreement for all types of loading histories, allows calculating in a simple manner the reduction on the strength for a long-term loading situation and also its associated deformation capacity. On that basis, a comprehensive method for predicting failure of concrete under different long-term loading patterns is proposed and validated.

3.2 Introduction

Many concrete structures are subjected to significant sustained loads over large periods during their lifetime. This is for example the case of large bridges, cut-and-cover tunnels (*Fig. 3-1*a) and soil-retaining structures. Others are designed to carry mostly rapid load actions typically associated to traffic loads (as underpasses (*Fig. 3-1*b) and small bridges), wind or earthquakes.

In the last decades, large research efforts have been dedicated to understand the influence of the load type (sustained, variable) on the behaviour of concrete structures. The influence of sustained loads on the time-dependent deformation of structures in the serviceability state has also been widely researched. However, the influence of high levels of stress but variable with time and its structural consequences (stress redistributions for instance) is still poorly investigated and understood.

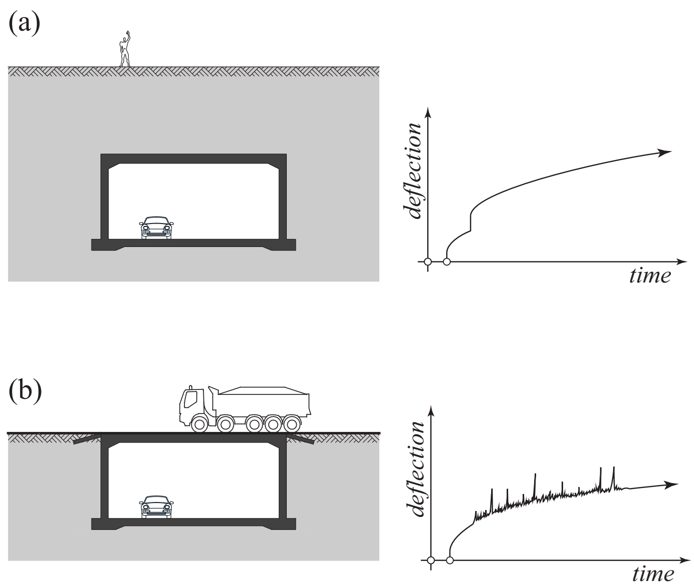

Fig. 3-1 Influence of sustained and rapid loads on structures: (a) cut-and-cover tunnel under permanent soil load; and (b) underpass under traffic load

The phenomenon of creep has traditionally been related to serviceability limit states and hence it has been mostly studied in the elastic domain of response of concrete, normally assumed to be up to about 40 % of its short term resistance. The first studies on compressive creep behaviour above this threshold (Davis 1928; Troxell et al. 1958) were still performed for moderate stress levels (up to 50% of the short-term strength of concrete), but already indicated that an amplification of creep strains occurs at high sustained loads (“*the creep per unit stress at the higher levels of stress intensity was significantly greater*” (Troxell et al. 1958)). With respect to the age of loading, it was observed a lower influence of the level of stress on the creep behaviour for concretes loaded at older ages. Other authors (Freudenthal and Roll 1958; Roll 1964) performed extensive studies of concrete under sustained compressive loads for various concrete mixes, showing that the strength increase with age depended also on the specimen geometry (influencing the drying conditions). They also concluded that there is in general an increase of strength for specimens subjected to sustained loading. However, it is important to mention that their specimens were loaded with sustained stress levels up to 60 % of the 28-day short term strength, and then loaded rapidly to failure on the day of removing of sustained load.

With respect to very high levels of sustained compressive stress (close to the short-term strength), Shank (Shank 1949) and Rüsch (Rüsch 1956, 1960; Rüsch et al. 1968) were amongst the first authors to investigate the existence of a delayed failure under sustained loading. In 1960, Rüsch established the term “*sustained load strength*” (Rüsch 1960) for the critical stress level leading to failure under sustained loading, which was found to be approximately 75 % of the short term strength. That research work established a failure limit for concrete under sustained load, governing for stress levels above that critical degree of loading (see *Fig. 3-2*a). For a sustained load below the critical degree of loading, concrete would on the contrary reach the so-called creep limit (strains increase with time by creep deformations but not leading to failure). This pioneer work was followed by several researchers, some of which have studied the failure under compressive creep of normal strength concrete (Shah and Chandra 1970; Awad and Hilsdorf 1971; Diaz and Hilsdorf 1971; Stöckl 1972; Wittmann and Zaitsev 1974; Coutinho 1977; Smadi et al. 1985; Fouré 1985; Nechvatal et al. 1994; El-Kashif and Maekawa 2004; Maekawa and El-Kashif 2004; Fernández Ruiz et al. 2007; Suryanto et al. 2013; Wang et al. 2015; Schlappal et al. 2017) as well as of high strength concrete (Ngab et al. 1981; Smadi 1983; Smadi et al. 1985; Han and Walraven 1994; Müller et al. 2010; Anders 2012). Several authors have even provided results on the influence of sustained loads on the multiaxial behaviour and Poisson’s ratio (Bažant 1970; Zhaoxia 1994; Benboudjema et al. 2001; Mazzotti and Savoia 2002; Aili et al. 2015).

Table 3-1 gives an overview of other experimental works on sustained load strength of concrete in compression. In accordance to the findings of Rüsch, most of the researchers have found that failure under sustained load occurs for normal strength concrete at approximately 70 % to 75 % of its short-term strength. Completing these researches, several works have also been performed on the phenomenon of sustained load strength for low compressive loading rates (El-Kashif and Maekawa 2004; Fernández Ruiz et al. 2007; Fischer et al. 2014; Tasevski et al. 2015, 2016) as well as for flexural and tensile loading (Domone 1974; Reinhardt and Cornelissen 1985; Zhou 1992; Carpinteri et al. 1997; Rinder 2003; Barpi and Valente 2005; Reinhardt and Rinder 2006; Omar et al. 2009).

Table 3-1 Experimental programmes performed in the scientific literature for failure of concrete in compression under sustained loading

Reference	$f_{c,ref}\,(t_0)$ [MPa]	Observed thresh. $f_{c,\Delta t}\,/\,f_{c,ref}\,(t_0)$ [-]	t_0 [d]	Tested Δt up to: [-]	Tested $f_{c,\Delta t}\,/\,f_{c,ref}\,(t_0)$ [-]	Failure Δt [-]
Sell, 1959	32	0.70	56	7 d	-	-
Rüsch, 1960	20 - 60	0.75	var.	2 y	0.75	up to 6 months
					0.80	up to 3 months
					0.85	up to several days
					0.90	up to several hours
Awad, 1971	16 - 41	-	var.	4 d	0.85	up to 4.5 days
					0.90	up to 7 hours
					0.95	up to 1.5 hours
Stockl, 1972	50	0.80	28	15 y	0.70-0.75	several failures reported
Smadi et al, 1982	20 - 25	0.75	28	6 m	-	-
	35 - 40	0.75			0.75	2 fail at 49 days, 2 no fail
	60 - 70	0.80			0.80	2 fail at 14 days, 2 no fail
Iravani et al, 1998	65	0.70-0.75*	56	3 m	0.75/0.80*	fail at 30 days / 7 days
	95	0.75-0.80*			-	-
	105	0.85-0.90*			0.90/0.95*	fail at 0.5 days / 9 min
	120	0.85-0.90*			-	-

*slightly eccentric

Regarding the kinematics of creep strains under high constant stresses, three stages of behaviour can generally be distinguished in the development of creep as mentioned by several authors (Zhou 1992; Berthollet et al. 2004; Bockhold and Stangenberg 2004), namely primary, secondary and tertiary creep (Fig. 3-2b). The primary creep relates to a kinematics associated to a decreasing rate of strains ($\ddot{\varepsilon} < 0$). The secondary creep is characterized by a roughly constant rate of strains ($\ddot{\varepsilon} \sim 0$), whereas the tertiary creep is associated to an increasing rate of strains ($\ddot{\varepsilon} > 0$, eventually leading to failure). The development of secondary and tertiary creep strains is highly dependent on the stress level (σ_c/f_c) to which concrete is subjected. With this respect, several phenomenological discussions can be found in recent literature (Rossi et al. 1994, 2012, 2013; Bažant and Xiang 1997; Berthollet et al. 2004; Denarié et al. 2006; Fernández Ruiz et al. 2007). In general, linear creep strains are associated to delayed strains occurring in concrete without material damage (low σ_c/f_c ratios). For higher stress levels, nonlinear creep strains develop due to microcrack initiation and progress in the concrete, implying material damage (Rossi et al. 2013). Under some conditions (high sustained σ_c/f_c ratios, normally above 0.75), the tertiary creep stage may develop, characterized by microcrack progression and coalescence (creation of macrocracks), which is accompanied by additional nonlinear creep strains and eventually leads to failure under sustained load.

With respect to codes of practice, the topic of nonlinear creep strains is in some cases explicitly considered as in *fib* Model Code 2010 (fib 2013b) or in Eurocode 2 (CEN 2004), where formulas are provided to estimate the linear and nonlinear creep strains for ratios σ_c/f_c normally lower than 0.6 (prior to the development of tertiary creep strains and failure under sustained load). These codes also account for the reduction of strength due to high sustained loads (ratios $\sigma_c/f_c > 0.75$ according to *Fig. 3-2*a), although this effect may often be neglected in the design of new structures as it is potentially compensated by the strength increase with concrete age. Despite these indications, no guidelines are normally provided on how to calculate the strains at failure when tertiary creep develops (necessary to calculate potential stress redistributions and system strength) or on how to evaluate the response of concrete for variable stress levels implying nonlinear creep strains (potentially influencing the final material strength).

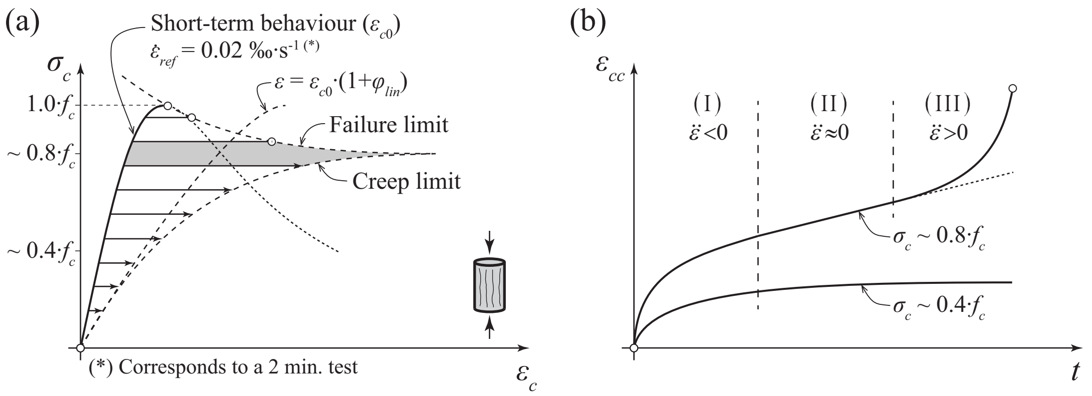

Fig. 3-2 Response of concrete under uniaxial compressive stresses: (a) short- and long-term stress–longitudinal strain diagram according to (Rüsch 1960); and (b) evolution of creep strains with time (primary, secondary and tertiary stages of creep)

Within this frame, the necessity of more comprehensive, consistent and realistic models to consider the long-term response and strength of concrete is gaining importance in the last years. This is justified to a large extent by the assessment of existing structures, where the concrete strength is updated at a concrete age where almost no further strength increase can be expected and high levels of sustained load may occur (even after some years of construction due to additional dead loads originated for instance by refurbishing or addition of new stories in buildings or to placing of new asphalt layers in bridges). To contribute to this topic, this paper presents the results of a comprehensive investigation on the influence of high levels of sustained stresses on the concrete compressive strength and deformation capacity, with particular focus on variable stress histories. The main aim is to provide a general and consistent framework to address the linear and nonlinear creep strains and to calculate the progression of material damage and delayed failure under a general loading history. To that aim, the results of a specific experimental programme are presented and discussed. Finally, the experimental results are analysed by extending a theoretical approach previously developed by the authors (Fernández Ruiz et al. 2007). The model is shown to accurately predict both the strength and the deformation capacity at failure and to allow investigating on the various parameters influencing the phenomenon.

3.3 Development of linear and nonlinear creep strains in concrete

3.3.1 Types of time-dependent strains under sustained loading

This section presents a brief summary of the state of knowledge on the uniaxial concrete behaviour under constant sustained loads that will later be extended in this paper. To that aim, creep strains will be considered as the strains developed in time in excess of those associated to shrinkage (Rüsch 1960).

At sustained stress levels lower than approximately $0.4 \cdot f_c$, there is an almost linear correlation between the delayed creep strains and the short-term strains (*Fig. 3-2*a). The behaviour under sustained stress consists in this case of primary creep, where the creep rate is high at the beginning and progressively decreases with time (*Fig. 3-2*b). At stress levels higher than approximately $0.4 \cdot f_c$, the damage process in the material starts. A part of the damage process occurs during the application of the load (microcrack generation) and another part during the stage of sustained loading (mostly due to microcrack propagation). The linear correlation between the delayed creep strain and the elastic strain is lost and additional nonlinear creep strains develop.

The second threshold (*Fig. 3-2*a, shaded area) describes the stress level above which delayed failures are possible. At stress levels higher than $\sim 0.75 \cdot f_c$, microcrack coalescence may develop giving rise to the onset of the tertiary creep phase. This phase is characterized by an increasing rate of delayed deformations, with a final uncontrolled process of progressive crack coalescence leading to failure (stage III in *Fig. 3-2*b). It is worth noting that the value of the second threshold may vary between 0.75-0.80 for normal strength concrete and 0.80-0.85 for high strength concrete, according to the literature reported in *Table 3-1*.

3.3.2 Failure under sustained load – inelastic strain capacity

As already well established for rocks (Goodman 1989), failure as a result of creep in compression can be predicted based on the stress-strain curve. According to Goodman (Goodman 1989), for high levels of sustained stress, the creep process leads to failure if the accumulated inelastic strain developed within the creep process intersects the descending branch of the stress-strain curve. With respect to concrete, many authors (see for instance (Karsan and Jirsa 1969; Zhou 1992)) have also used the monotonic stress-strain curve of concrete as a failure criterion in the case of cyclic loading.

Consistently with these observations, and following the approach of (Fernández Ruiz et al. 2007), it can be assumed that the inelastic strain capacity for a given stress level is equal to the difference between the instantaneous post- and pre-peak strains for that level of stress, see *Fig. 3-3*a. This value can thus be directly calculated by using the monotonic stress-strain curve of concrete in compression, refer to *Fig. 3-3*a,b where a reference strain rate $\dot{\varepsilon} = 0.02$ ‰$\cdot$s^{-1} is usually considered to characterize the monotonic response (approximately 100 seconds to reach the maximum strength). This approach has shown to yield consistent results (Fernández Ruiz et al. 2007) and its validity will later be investigated and confirmed by the results presented in this paper.

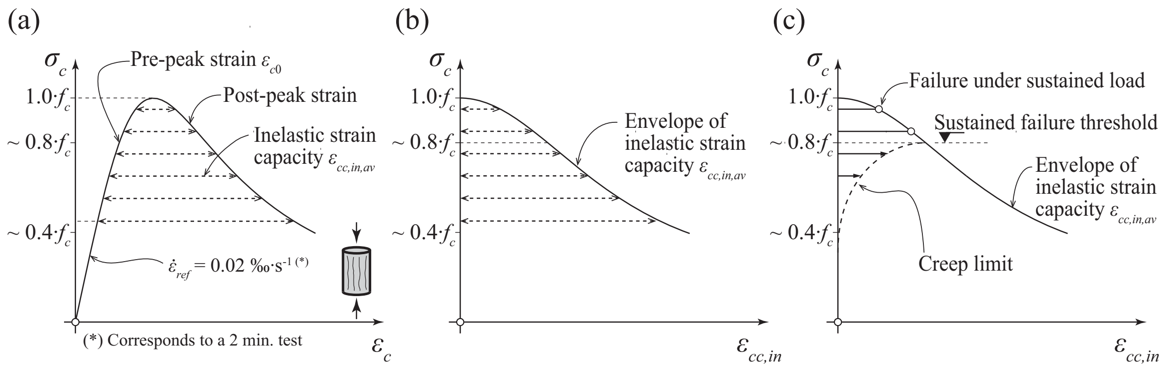

Fig. 3-3 Inelastic strain capacity: (a) instantaneous pre- and post-peak longitudinal strains; (b) definition of the inelastic strain capacity as difference between instantaneous post- and pre-peak longitudinal strains; and (c) failure limit and creep limit

The inelastic strain developed within a process of sustained loading can on the other hand be calculated by removing the pre-peak instantaneous strain, the shrinkage strains and the linear creep strains from the total measured strain. If the stress level in a sustained load test is high enough ($\sigma_c/f_c \geq 0.75$), the inelastic strains developed in the concrete may reach the strain capacity and failure occurs, refer to *Fig. 3-3*c. Otherwise, for moderate or low levels of stress, the inelastic strain capacity is not attained (creep limit, *Fig. 3-3*c). In such case, the load in the specimen can still be increased until failure (Fernández Ruiz et al. 2007).

3.3.3 Analysis of creep effects based on the affinity assumption between linear and nonlinear (secondary) creep strains

Fernández Ruiz et al. presented in a previous work (Fernández Ruiz et al. 2007) a framework for predicting the nonlinear creep strains of concrete under high levels of sustained stress. That method was originally developed only for constant sustained loads. The approach was based on the assumption that the strains developed over time *t* in a concrete structure loaded at time t_0 can be described by following expression:

$$\varepsilon_c\left(t,\frac{\sigma_c}{f_c}\right) = \varepsilon_{c0}\left(t_0,\frac{\sigma_c}{f_c}\right) + \Delta\varepsilon_{cs}(t,t_0) + \Delta\varepsilon_{cc}\left(t,t_0,\frac{\sigma_c}{f_c}\right) \tag{3.1}$$

where the first term on the right side of the equation ($\varepsilon_{c0}\left(t_0,\frac{\sigma_c}{f_c}\right)$) corresponds to the instantaneous pre-peak strain (including elastic and inelastic components), the second ($\Delta\varepsilon_{cs}(t,t_0)$) to the shrinkage strains and the third ($\Delta\varepsilon_{cc}\left(t,t_0,\frac{\sigma_c}{f_c}\right)$) to the creep strains.

The shrinkage strains are considered (according to their definition) to be independent from the material stress state. The creep strains, on their turn, are related to the pre-peak strains by means of a creep coefficient φ which depends on the time at loading, the duration of the action and the level of stress:

$$\Delta\varepsilon_{cc}\left(t,t_0,\frac{\sigma_c}{f_c}\right) = \varepsilon_{c0}\left(t_0,\frac{\sigma_c}{f_c}\right)\cdot\varphi\left(t,t_0,\frac{\sigma_c}{f_c}\right) \tag{3.2}$$

The creep coefficient can be assumed to be stress independent for sustained stress levels below $0.4{\cdot}f_c$ ($\varphi = \varphi_{lin}(t,t_0)$). Above this threshold, the creep coefficient is no longer stress independent. For its calculation, for stress levels σ_c/f_c below 0.7 (i.e. where no failure under sustained loading occurs), a simple formula was proposed by (Fernández Ruiz et al. 2007) assuming perfect affinity between linear and nonlinear creep strains:

$$\varphi_{nl}\left(t, t_0, \frac{\sigma_c}{f_c}\right) = \eta\left(\frac{\sigma_c}{f_c}\right) \cdot \varphi_{lin}(t, t_0) \tag{3.3}$$

where, for design purposes, the coefficient $\eta(\sigma_c/f_c)$ can be estimated as (Fernández Ruiz et al. 2007):

$$\eta\left(\frac{\sigma_c}{f_c}\right) = 1 + 2\left(\frac{\sigma_c}{f_c}\right)^4 \tag{3.4}$$

The validity of Eq. (3.4) was demonstrated at long-term and for stress levels $\sigma_c/f_c < 0.7$, showing a satisfactory agreement to test results (Fernández Ruiz et al. 2007). Beyond the threshold of validity of Eq. (3.4) (stress levels σ_c/f_c above 0.7, where tertiary creep strains potentially develop), Fernández Ruiz et al. proposed, based on existing experimental data, that the total inelastic strain at sustained load failure is roughly composed by ⅔ of the nonlinear strain according to Eq. (3.4) and ⅓ of additional strains due to tertiary creep (Fernández Ruiz et al. 2007). These considerations will later be discussed more in detail in this paper.

After publication of that work (Fernández Ruiz et al. 2007), the authors found a similar approach to that of Eq. (3.4) in the old USSR code of practice (USSR 1987) based on the following expression (original notation modified to be consistent to the one of this paper):

$$\eta\left(\frac{\sigma_c}{f_c}\right) = 1 + V_t\left(\frac{\sigma_c}{f_c}\right)^m \tag{3.5}$$

where $V_t = V_0{\cdot}e^{-f}$ and $f = f(t\text{-}t_0) = k{\cdot}e^{-\gamma\cdot(t-t_0)}$. The values of V_0, k, γ and m are given in the USSR code for different strength classes of normal hardening concrete, but do not differ much from those of Eq. (3.4). It is interesting to note that the approach is very similar, and yields comparable results which confirm the plausibility (and practical interest) of the approach.

In this article, the validity of the affinity hypothesis for the entire progress of tertiary creep development is examined, and a generalization for different load cases is proposed.

3.4 Experimental programme

The aim of this experimental programme is to study the response of normal strength concrete at high stress levels both at varying strain and stress rates.

3.4.1 Materials and testing methods

The uniaxial compressive behaviour of concrete under different strain and stress rates has been investigated by means of cylindrical specimens with dimensions Ø × h = 160 × 320 mm. The concrete was produced with a CEM-II 42.5R cement (w/c = 0.56) and Rhone river aggregates. The specimens were kept moulded (with the top face sealed) until the age of 21 days. Thereafter, the specimens were unmoulded and stored under standard laboratory conditions (temperature of 21 °C and relative humidity of 65 %) until testing. The tests were performed using a Schenck Hydroplus servo-hydraulic testing machine (*Fig. 3-4*a) with capacity of 2.5 MN and a custom-made steel frame which enhances the stiffness of the test setup (Fernández Ruiz et al. 2007). The climatic room where the tests were performed has controlled temperature (21±0.5 °C) and relative humidity (65±3 %, some minor deviations are commented later). The longitudinal strain of the specimens $\varepsilon_{c,long}$ was measured with three surface displacement transducers (omega gauges) arranged radially on two steel rings at a distance of 250 mm (see *Fig. 3-4*b). The signal of the three transducers was acquired at high frequency (1200 Hz) and it was used to control the strain rate (in the strain rate test series) directly on the specimen. The transverse strain $\varepsilon_{c,trans}$ was measured with a steel ribbon dilatometer (*Fig. 3-4*b) equipped with a linear variable differential transformer (LVDT).

The reference compressive strength was tested at a strain rate of 0.02 ‰·s^{-1}, corresponding to approximately 100 seconds before maximum strength is reached. Its development with concrete age has been compared to the Model Code 2010 (fib 2013b) formula:

$$f_{c,ref}(t_0) = f_{c,28} \cdot e^{s \cdot \left[1-\sqrt{\frac{28}{t}}\right]} \tag{3.6}$$

where $f_{c,28}$ = 29.0 MPa and t refers to the concrete age in days. The coefficient s was adapted by means of least square fitting and returns the value $s = 0.316$. The resulting development curve is given in *Fig. 3-5*a.

With respect to the shrinkage strains, a shrinkage rig with three cylindrical specimens was installed after unmoulding of the specimens (at 21 days), see *Fig. 3-4*c. The shrinkage measurements were performed during two years. Then, the standard shrinkage model from Model Code 2010 was used to reproduce the measured shrinkage strains (with α_{as}=600, α_{ds1}=6 and α_{ds2}=0.012 corresponding to a cement class 42.5 R). The resulting expression is compared in *Fig. 3-5*b to the test results:

$$\varepsilon_{cs}(t, t_s) = -36.4 \cdot 10^{-6} \cdot \left(1 - e^{-0.2 \cdot \sqrt{t}}\right) - 699 \cdot 10^{-6} \cdot \frac{1}{\sqrt{\frac{224}{t - t_s} + 1}} \tag{3.7}$$

where t_s is the concrete age at demoulding in days. Measurements on linear creep were also performed during two years in a standard creep rig with three cylindrical specimens loaded at a stress level $\sigma_c/f_c \approx 0.35$ at an age of 28 days (*Fig. 3-4*d). The creep strains were calculated by removing from the increase of strains with time the value corresponding to the measured shrinkage strains. Then, the standard creep law from Model Code 2010 was adapted by fitting the parameters to reproduce the measured curve (*Fig. 3-5*c):

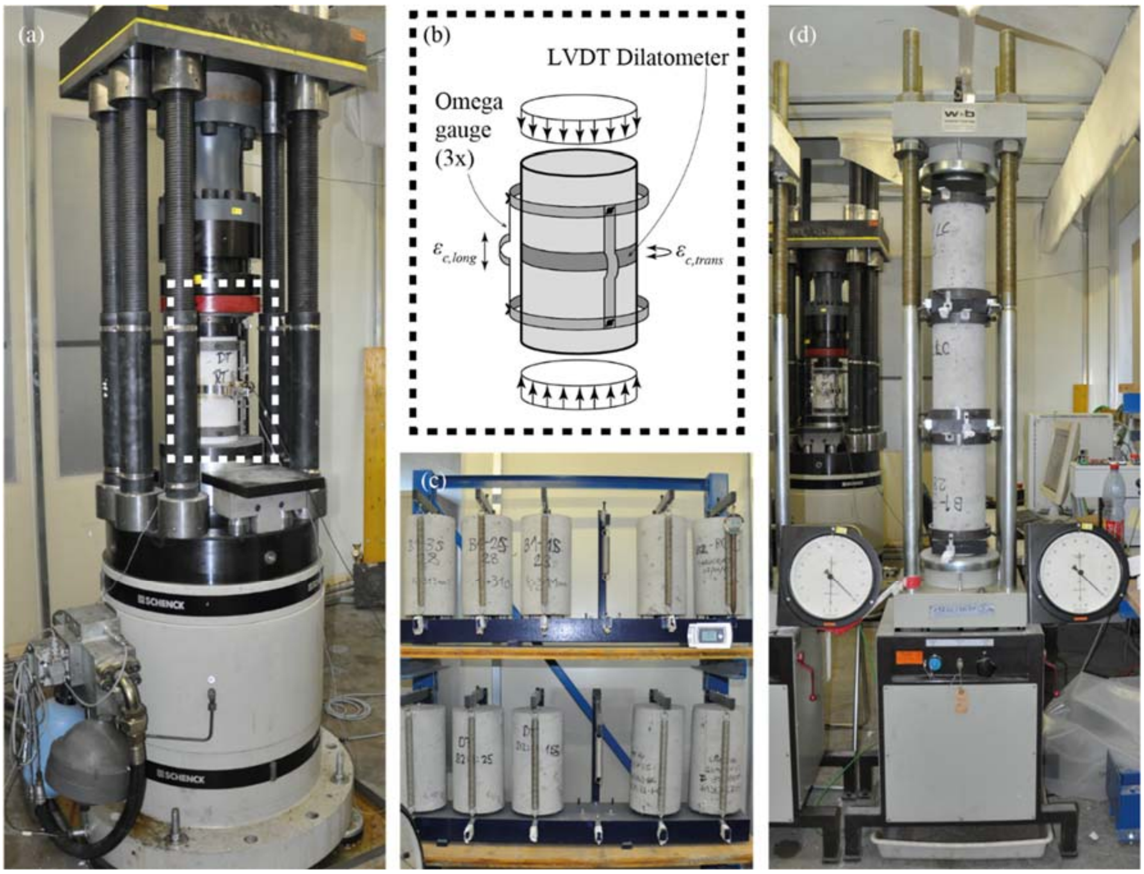

Fig. 3-4 Test setup: (a) testing frame for stress and strain rate long-term testing; (b) details of measurement devices; (c) shrinkage rigs; and (d) linear creep rigs

$$\varphi(t,t_0) = 3.24 \cdot \frac{1}{0.1 + {t_0}^{0.2}} \cdot \left[\frac{t - t_0}{682 + t - t_0}\right]^{\frac{1}{2.3+\frac{3.5}{\sqrt{t_0}}}} + 3.00 \cdot \frac{1}{0.1 + {t_0}^{0.2}} \cdot \left[\frac{t - t_0}{395 + t - t_0}\right]^{\frac{1}{2.3+\frac{3.5}{\sqrt{t_0}}}} \tag{3.8}$$

where t_0 is the age of concrete at loading. It can be noted that the results obtained with the Model Code 2010 formulae slightly underestimate the linear creep strains and slightly overestimate the shrinkage strains up to the age of approximately 400 days, being fairly accurate thereafter. In any case, the delayed strains (linear creep and shrinkage) are reasonably well estimated by this model. The linear creep law for other loading ages was also calculated according to the Model Code 2010 expression for the creep coefficient (the two grey lines in *Fig. 3-5*c represent the calculated linear creep coefficients for the average loading ages of the two experimental series).

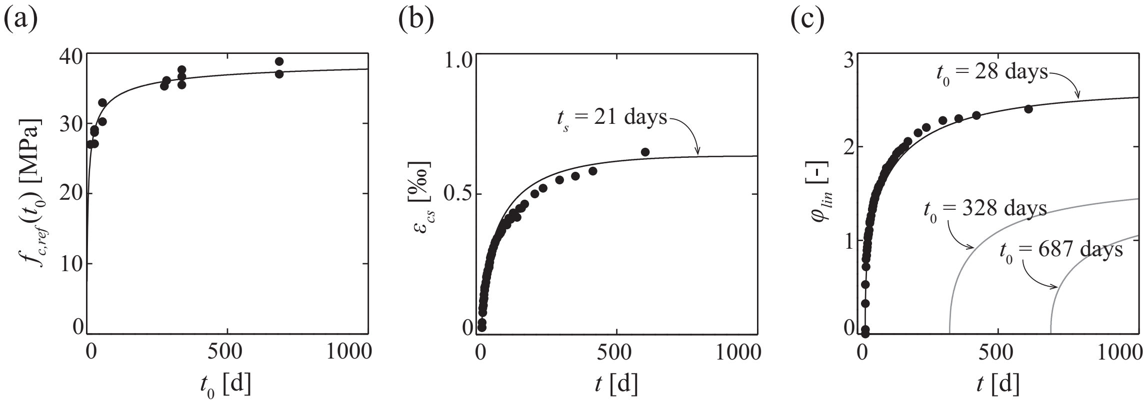

Fig. 3-5 Time-dependent concrete properties: (a) development of compressive strength with time (tests performed at reference loading rate), (b) longitudinal shrinkage strains and (c) longitudinal linear creep strains ($\sigma_c/f_c \approx 0.35$)

It shall be commented that the relative humidity was kept as constant as possible. However, there have been some variations (±10 %) over very limited periods of time during the two years, mainly due to maintenance operations of the climate regulation system. These variations have been taken into account for the shrinkage and linear creep calculations presented in *Fig. 3-5*b and *Fig. 3-5*c according to Model Code 2010.

3.4.2 Types of loading

The delayed behaviour of concrete under uniaxial compression has so far been usually investigated with classical creep tests where a constant stress level is sustained over time (Rüsch 1960; Fernández Ruiz et al. 2007). To investigate other long-term loading patterns, two new test series were performed in this study.

The first test series was performed by varying the strain rate (see *Fig. 3-6*). To have a fairly mature concrete and to limit the influence of strength increase with time, the concrete age at testing varied between 10 and 14 months (for more details on the evolution of concrete strength with time refer to *Fig. 3-5*a and *Table 3-2*). This test series covered strain rates ranging from $2.00 \cdot 10^{-3}$ s^{-1} to $2.00 \cdot 10^{-9}$ s^{-1} (time to failure from 1 second to ca. 14 days).

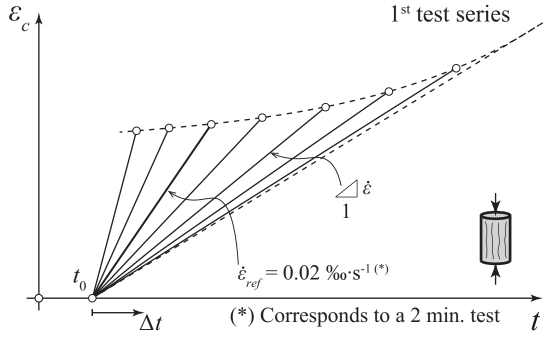

Fig. 3-6 Loading paths of 1st test series (DR tests): varying longitudinal strain rate

The second test series of uniaxial compression was performed by varying the stress rate. The concrete age at testing varied between 22 and 24 months (refer again to *Fig. 3-5*a and *Table 3-3*). First, an initial loading ramp ($3.50\cdot10^{-1}$ MPa·s^{-1}) was applied up to 80 % of the reference strength ($\dot{\varepsilon} = 0.02$ ‰·s^{-1}) at the day of loading (see *Fig. 3-7*). Then, a second loading ramp was applied from $5.00\cdot10^{0}$ MPa·s^{-1} to $1.25\cdot10^{-5}$ MPa·s^{-1} (time to failure from 1 second to ca. 5 days).

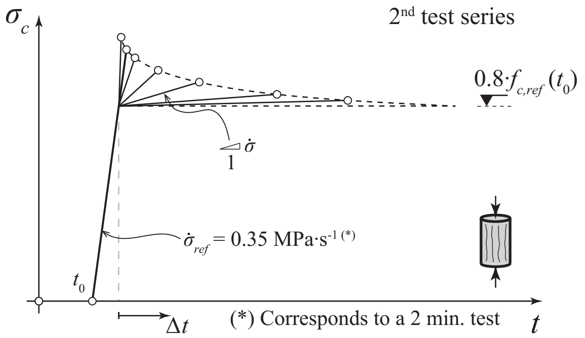

Fig. 3-7 Loading paths of 2nd test series (LR tests): varying stress rates at high stress levels

Despite the fact that the test series of this programme were aimed at investigating the influence of low stress and strain rates on the compressive strength of concrete, some tests with relatively high stress and strain rates were also conducted (tests DR1, DR2 and LR0). The results of these tests are presented in the following, but they will not be investigated in detail thereafter.

3.4.3 Results for strain rate tests (1st test series)

Table 3-2 presents an overview of the results of the test series performed with varying strain rate. The second column indicates the control strain rate, the third and fourth column give the age of concrete and the reference strength at load application respectively, and columns 5-9 give the main results (the average of every strain rate is written in bold font). The stress–strain diagrams are presented and compared to the reference test with $\dot{\varepsilon} = 0.02$ ‰·s^{-1} in *Fig. 3-8*. The evolution of the stress and of the strains versus time in logarithmic scale is also presented in *Fig. 3-9*a-c. It is worth noting that the specimen DR7_1 was tested with a very low strain rate and thus a preload had to be applied in order not to lose contact between the specimen and the loading plate (this explains the shape for stresses lower than $0.2\cdot f_{c,ref}(t_0)$ in *Fig. 3-9*).

These tests confirm that the failure load is lower as the strain rate decreases (load applied over longer periods of time), with a variation of the measured strength to the reference strength (standard loading speed) at the same age ranging from 1.10 (high strain rates) to 0.932 (low strain rates). Also, it can be observed that the longitudinal and transverse strains at failure usually increase with decreasing strain rates. In terms of effective Poisson's ratio ($\nu_{eff} = \varepsilon_{c,trans}/\varepsilon_{c,long}$), an increasing trend could be observed with average values from 0.223 for higher strain rates (standard value for concrete tested in displacement control) up to 0.375 for the lower strain rates.

Table 3-2 Overview of the results of the 1st test series performed with varying strain rate (rows marked in bold as “Avg.” indicating average of specimens with same loading rate)

Test Name	$\dot{\varepsilon}$	t_0	$f_{c,ref}(t_0)$	Δt	$f_{c,\Delta t}$	$f_{c,\Delta t} / f_{c,ref}(t_0)$	$\varepsilon_{c,long}$	$\varepsilon_{c,trans}$
	[s^{-1}]	[d]	[MPa]	[s]	[MPa]	[-]	[‰]	[‰]
DR1_1	**2.00E-03**	**340**	**36.3**	**1.00E+00**	**40.1**	**1.10**	**2.16**	**0.548**
DR2_1	2.00E-04	340	36.3	9.20E+00	37.9	1.04	2.21	0.562
DR2_1	2.00E-04	340	36.3	9.40E+00	38.6	1.06	2.29	0.440
Avg. DR2	**2.00E-04**	**340**	**36.3**	**9.30E+00**	**38.3**	**1.05**	**2.25**	**0.501**
DR3_1 (ref.)	2.00E-05	339	36.3	9.40E+01	36.7	1.01	2.28	0.543
DR3_2 (ref.)	2.00E-05	339	36.3	9.78E+01	35.5	0.977	2.38	0.763
DR3_3 (ref.)	2.00E-05	339	36.3	9.74E+01	37.7	1.04	2.37	0.608
Avg. DR3	**2.00E-05**	**339**	**36.3**	**9.64E+01**	**36.6**	**1.01**	**2.34**	**0.638**
DR4_1	2.00E-06	284	36.0	1.02E+03	33.8	0.939	2.49	0.726
DR4_2	2.00E-06	280	36.0	1.09E+03	35.9	0.999	2.62	0.998
Avg. DR4	**2.00E-06**	**282**	**36.0**	**1.06E+03**	**34.9**	**0.969**	**2.56**	**0.862**
DR5_1	2.00E-07	276	36.0	1.16E+04	34.7	0.966	2.81	0.813
DR5_2	2.00E-07	278	36.0	1.21E+04	34.5	0.959	2.95	1.34
Avg. DR5	**2.00E-07**	**277**	**36.0**	**1.19E+04**	**34.6**	**0.962**	**2.88**	**1.08**
DR6_1	**2.00E-08**	**343**	**36.3**	**1.26E+05**	**34.1**	**0.939**	**3.09**	**0.956**
DR7_1	**2.00E-09**	**440**	**36.7**	**1.21E+06**	**34.2**	**0.932**	**2.94**	**0.396**

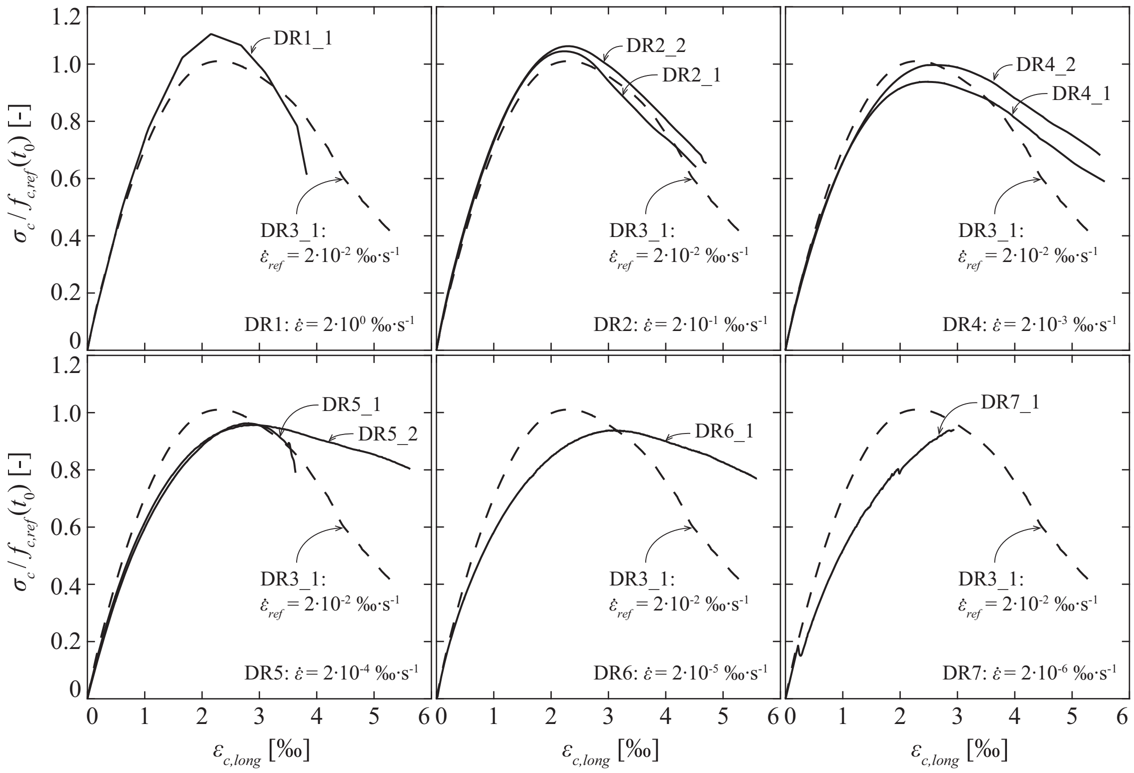

Fig. 3-8 Stress–strain diagrams of the 1st test series (tests with varying strain rate, refer to *Table 3-2*)

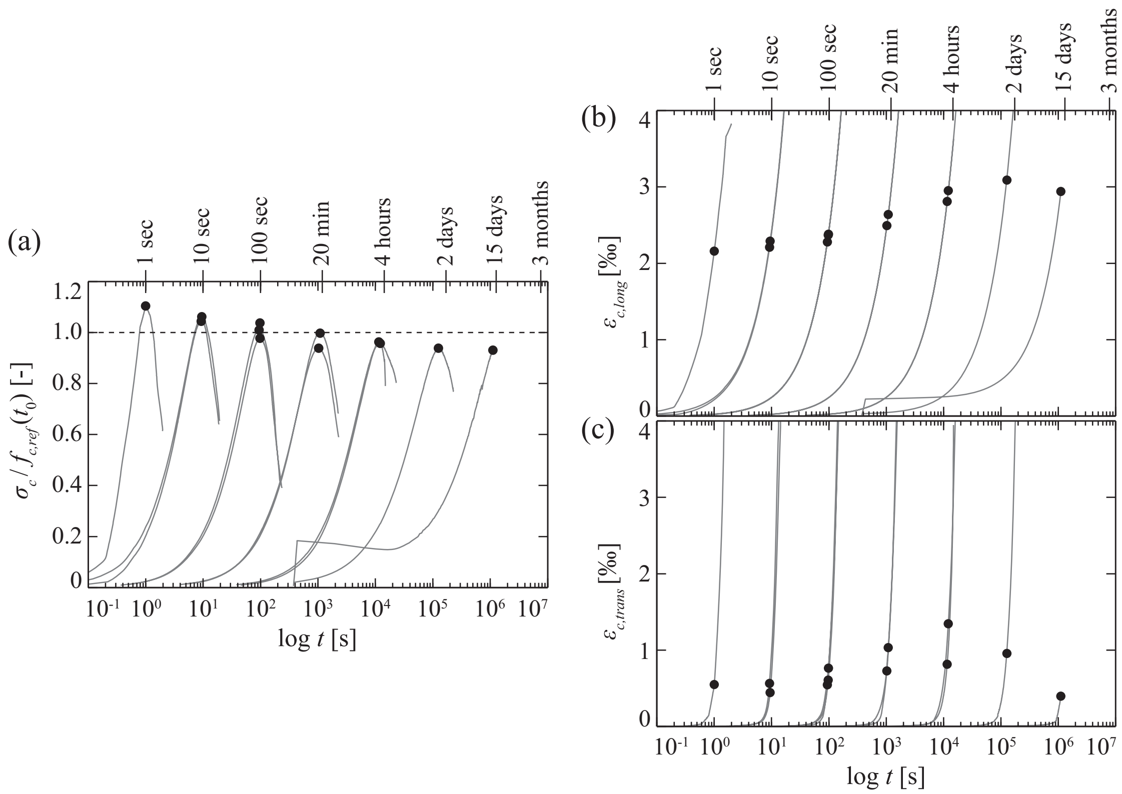

Fig. 3-9 Results of the 1st test series (tests with varying strain rate): (a) longitudinal stress; (b) longitudinal strain; and (c) transverse strain versus time

3.4.4 Results for stress rate tests (2nd test series)

Table 3-3 presents an overview of the results of the test series performed with varying stress rate. The second column indicates the control stress rate after the initial loading ramp, the third and fourth column give the age of concrete and the reference strength at load application respectively, and columns 5-9 give the main results (the average of every stress rate is written in bold font). The stress–strain diagrams are presented and compared to the reference test with $\dot{\varepsilon} = 0.02$ ‰·s^{-1} in *Fig. 3-10*. The evolution of the stress or the strains versus time in logarithmic scale is additionally presented in *Fig. 3-11*a-c.

As for the 1st series, it can be observed that the failure load is lower as the stress rate decreases (corresponding to longer times of application of the load, consistently to the strain rate tests). This decrease of the strength is accompanied by larger strains at failure in the longitudinal and transverse directions. Some fluctuations in the results can be observed, probably related to the load control of the test, where the failure occurs in a brittle manner and with no post-peak branch.

Table 3-3 Overview of the results of the 2nd test series performed with varying stress rate (rows marked in bold as "Avg." indicating average of specimens with same loading rate)

Test Name	$\dot{\sigma}$ [MPa·s^{-1}]	t_0 [d]	$f_{c,ref}(t_0)$ [MPa]	Δt [s]	$f_{c,\Delta t}$ [MPa]	$f_{c,\Delta t} / f_{c,ref}(t_0)$ [-]	$\varepsilon_{c,long}$ [‰]	$\varepsilon_{c,trans}$ [‰]
LR0_1	**5.00E+00**	**728**	**37.4**	**5.00E-01**	**38.9**	**1.04**	**2.27**	**-**
LR1_1	5.00E-01	684	37.3	3.19E+01	39.8	1.07	2.36	0.918
LR1_2	5.00E-01	684	37.3	2.89E+01	38.8	1.04	2.53	0.984
LR1_3	5.00E-01	684	37.3	2.39E+01	37.0	0.992	2.52	1.08
Avg. LR1	**5.00E-01**	**684**	**37.3**	**2.82E+01**	**38.5**	**1.03**	**2.47**	**0.994**
LR2_1	5.00E-02	678	37.3	1.27E+02	35.3	0.948	2.72	1.69
LR2_2	5.00E-02	679	37.3	2.28E+02	40.0	1.07	2.69	1.19
LR2_3	5.00E-02	679	37.3	1.87E+02	38.1	1.02	2.67	1.40
Avg. LR2	**5.00E-02**	**679**	**37.3**	**1.81E+02**	**37.8**	**1.01**	**2.69**	**1.43**
LR3_1	5.00E-03	678	37.3	1.38E+03	35.6	0.954	2.96	1.46
LR3_2	5.00E-03	682	37.3	1.71E+03	37.3	1.00	2.85	1.75
LR3_3	5.00E-03	682	37.3	1.53E+03	36.1	0.968	2.99	2.21
Avg. LR3	**5.00E-03**	**682**	**37.3**	**1.54E+03**	**36.4**	**0.974**	**2.93**	**1.81**
LR4_1	**1.50E-03**	**650**	**37.2**	**4.08E+03**	**34.7**	**0.933**	**2.94**	**2.27**
LR5_1	5.00E-04	682	37.3	1.33E+04	35.4	0.949	2.90	1.88
LR5_2	5.00E-04	683	37.3	1.60E+04	36.6	0.981	3.41	2.02
Avg. LR5	**5.00E-04**	**683**	**37.3**	**1.47E+04**	**36.1**	**0.965**	**3.16**	**1.95**
LR6_1	5.00E-05	684	37.3	1.43E+05	36.4	0.977	3.60	2.06
LR6_2	5.00E-05	707	37.3	1.82E+05	37.6	1.01	2.95	1.05
Avg. LR6	**5.00E-05**	**696**	**37.3**	**1.63E+05**	**37.0**	**0.992**	**3.28**	**1.56**
LR7_1	**1.25E-05**	**742**	**37.4**	**4.48E+05**	**34.2**	**0.914**	**3.28**	**1.35**

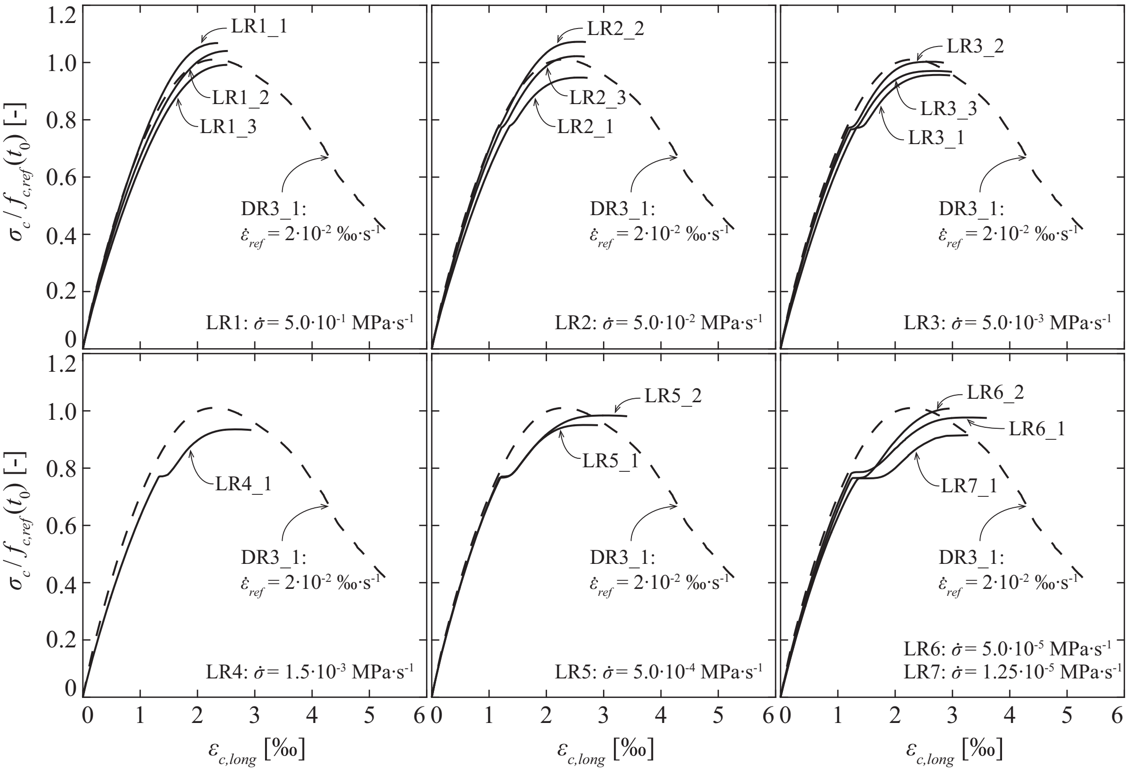

Fig. 3-10 Stress–strain diagrams of the 2nd test series (tests with varying stress rate, ref. to *Table 3-3*)

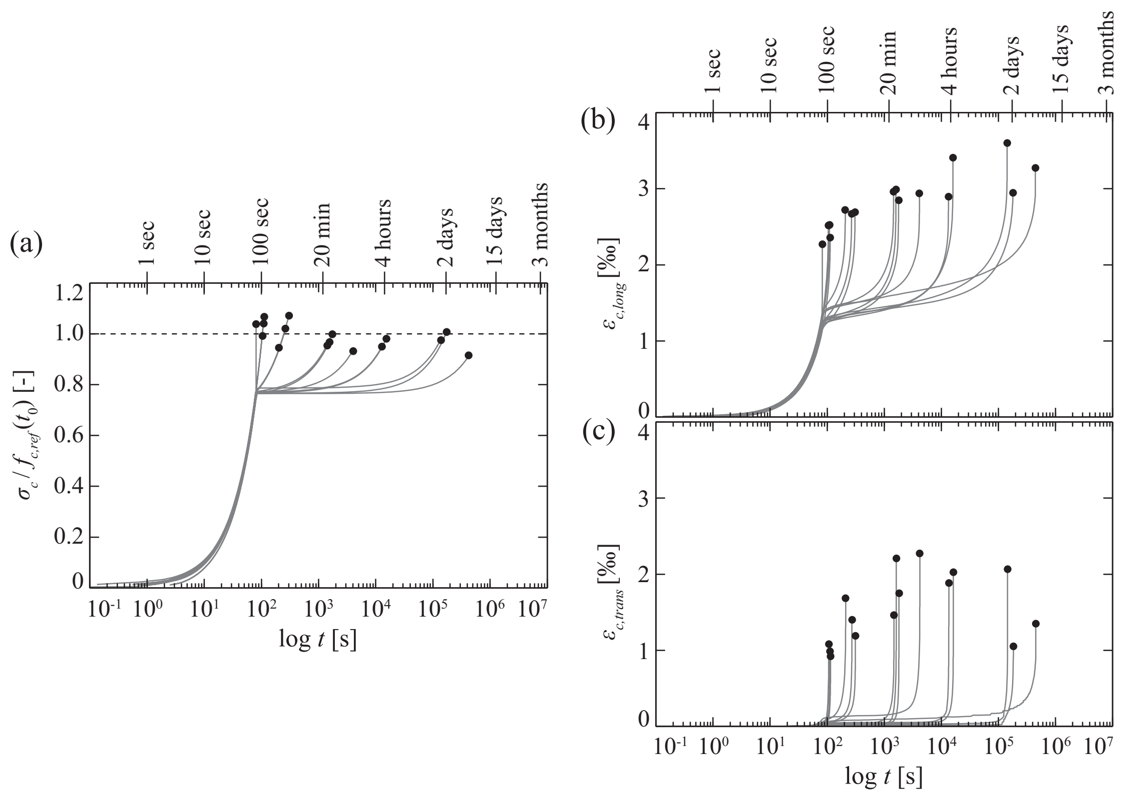

Fig. 3-11 Results of the 2nd test series (tests with varying stress rate): (a) longitudinal stress; (b) longitudinal strain; and (c) transverse strain versus time

In terms of the influence of the stress rate in the measured compressive strength of the material, it varies between 1.07 (high stress rates) and 0.914 (low stress rates) when it is normalized by the reference material strength (obtained with a standard loading speed). The effective Poisson's ratio ($\nu_{eff} = \varepsilon_{c,trans}/\varepsilon_{c,long}$), varies also from 0.402 for higher stress rates up to 0.617 for lower stress rates. These latter values (load control) are higher than those observed for strain rates (deformation control).

3.4.5 Discussion of inelastic strains developed at failure

1) Strain rate tests

As described in section 3.3.2, it has been observed for tests under constant sustained stress that when the developed nonlinear (inelastic) creep strain equals the inelastic strain capacity of the material for that level of stress, failure occurs ((Fernández Ruiz et al. 2007), refer to *Fig. 3-3*c). To evaluate the validity of this failure criterion for the investigated load patterns, the contributions of the instantaneous pre-peak strain (ε_{c0}), the linear creep strain ($\varepsilon_{cc,1}$) and the shrinkage strain (ε_{cs}) have been calculated and removed from the total strain:

$$\varepsilon_{cc,in} = \varepsilon_{tot} - \varepsilon_{c0} - \varepsilon_{cc,1} - \varepsilon_{cs} \tag{3.9}$$

To estimate the instantaneous pre-peak strains (ε_{c0}), the concrete model of (Fernández Ruiz et al. 2007) was used as a suitable stress-strain relationship reproducing the response of the reference tests (refer to section 3.11). As for the linear creep and shrinkage strains, the adapted laws from Model Code 2010 as defined in section 3.4.1 were used. The calculation of the creep strains due to variable stresses has been performed using the superposition principle described later in section 3.5. Finally, the estimated nonlinear (inelastic) creep strains were compared to the inelastic strain capacity, as presented in Fig. 3-12a. The figure shows a consistent agreement between the developed inelastic strain at failure and the inelastic strain capacity (difference between post- and pre-peak strains obtained as an average of three reference test curves with $\dot{\varepsilon} = 0.02$ ‰·s^{-1} or the analytical expression curve of Eq. (3.18)). This confirms the validity of the assumptions by (Fernández Ruiz et al. 2007) also for this loading pattern.

A similar analysis has been performed for the results of a previous test series performed by the authors (Tasevski et al. 2015, 2016) by using the same loading pattern (variable strain rate). The comparison is presented in *Fig. 3-12*c and shows also an excellent agreement.

2) Stress rate tests

In a similar manner as for the strain rate tests, the contributions of the instantaneous pre-peak strains, linear creep strains and shrinkage strains have been estimated and removed from the total strain recorded in the 2nd test series (refer to Eq. (3.9)). On that basis, the estimated nonlinear (inelastic) creep strain is compared to the inelastic strain capacity, as presented in *Fig. 3-12*b.

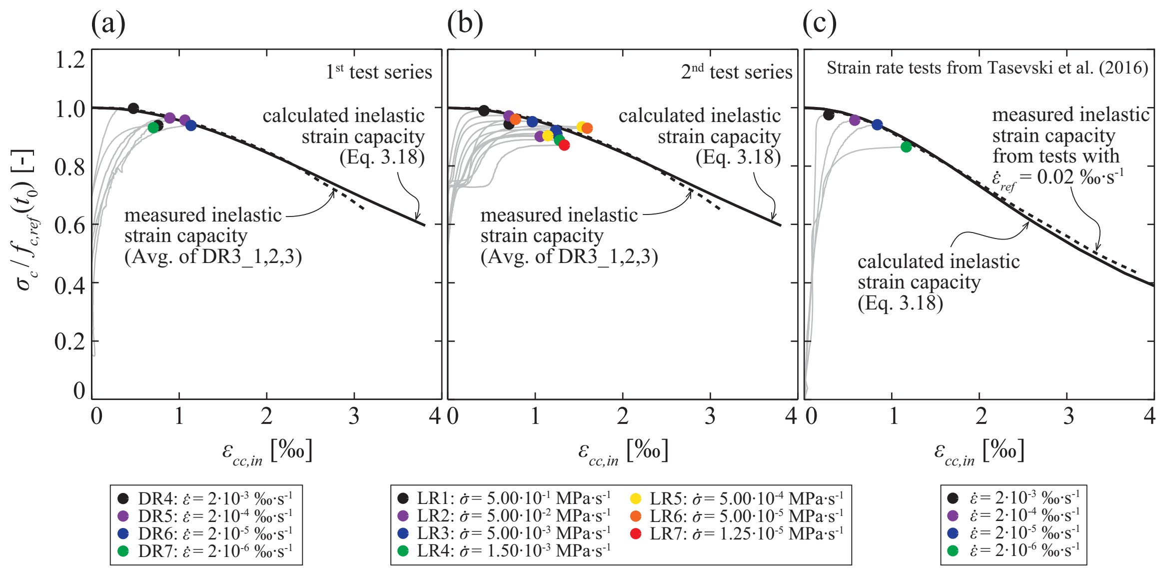

Fig. 3-12 Inelastic strains and corresponding inelastic strain capacity for: (a) 1st test series with constant strain rate, (b) 2nd test series with constant stress rate and (c) strain rate tests from (Tasevski et al. 2016)

As for the strain rate tests, an excellent agreement is found at failure between the inelastic strain capacity predicted by the reference curve and the nonlinear creep strain developed by the specimen for the various load patterns investigated, validating again the pertinence of this assumption.

3.5 Analysis of linear and nonlinear creep strain development

In this section, a simple method allowing to estimate the development of nonlinear secondary and tertiary creep strains is presented. The aim of this approach is to establish a stress–inelastic strain law based on the affinity hypothesis of creep strains (Fernández Ruiz et al. 2007) and applicable to any potential loading history. As shown in *Fig. 3-13*a, by intersecting this law with the failure criterion of inelastic strain capacity presented in section 3.3.2 (and confirmed by the experimental results in section 3.4.5), one can directly calculate the failure stress, the failure strain (deformation capacity) and the associated time to failure.

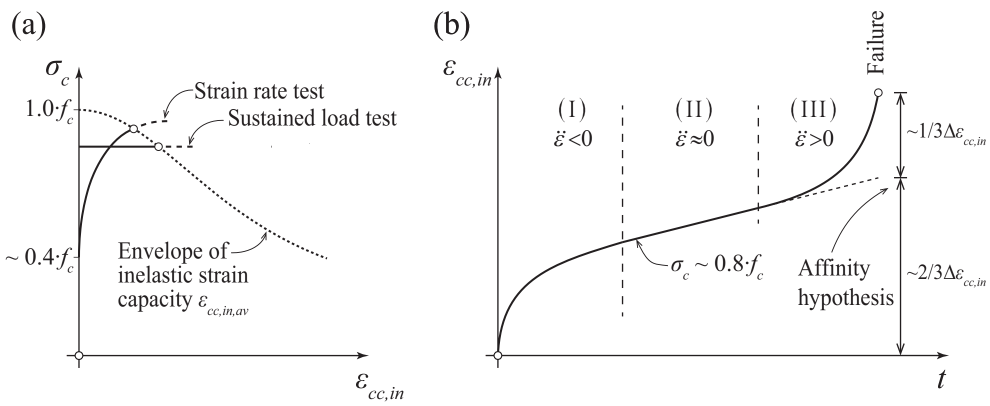

Fig. 3-13 Development of inelastic strains with time: (a) failure calculated by intersection of the stress-inelastic strain curve and the failure criterion of the inelastic strain capacity; and (b) inelastic

creep strain development with time based on the affinity hypothesis for a sustained load test, adapted from (Fernández Ruiz et al. 2007)

For extension of the framework proposed in (Fernández Ruiz et al. 2007), a distinction between primary, secondary and tertiary creep strains is performed in the following manner:

$$\varepsilon_{cc}\left(t, t_0, \frac{\sigma_c}{f_c}\right) = \varepsilon_{cc,1}(t, t_0) + \varepsilon_{cc,2}\left(t, t_0, \frac{\sigma_c}{f_c}\right) + \varepsilon_{cc,3}\left(t, t_0, \frac{\sigma_c}{f_c}, \frac{\varepsilon_{in}}{\varepsilon_{in,av}}\right) \tag{3.10}$$

where the primary creep strains are calculated by means of the linear creep coefficient:

$$\varepsilon_{cc,1}(t, t_0) = \varphi_{lin} \cdot \varepsilon_{c0} \tag{3.11}$$

The secondary creep strains (see section 3.3.1) are evaluated as (Fernández Ruiz et al. 2007):

$$\varepsilon_{cc,2}\left(t, t_0, \frac{\sigma_c}{f_c}\right) = (\eta - 1) \cdot \varepsilon_{cc,1}(t, t_0) \tag{3.12}$$

where the coefficient η is defined in a similar manner as Eq. (3.4), but the expression is generalized as follows:

$$\eta\left(\frac{\sigma_c}{f_c}, t, t_0\right) = \left(1 + 2 \cdot \eta_\tau(t, t_0)\left(\frac{\sigma_c}{f_c(t)}\right)^4\right) \tag{3.13}$$

In this expression, the coefficient η_τ accounts for the development of the nonlinear creep strains with time and can be calculated as:

$$\eta_\tau(t, t_0) = \left(1 - \log\left(\frac{t - t_0}{t_m + t - t_0}\right)\right)^n \tag{3.14}$$

The parameters t_m and n of this expression have been calibrated with the test results from the current study as well as from other literature (as later explained in section 3.6.2) and can be assumed as the following constant values: t_m = 100 days and n = 0.75. It can be seen by comparing Eqs. (3.4) and (3.13) that when $t \rightarrow \infty$, both equations yield the same result and thus the coefficient η_τ is mostly influencing at early ages. The pertinence of this improvement will be justified in section 5 by comparison to available test results (details on this phenomenon can be consulted elsewhere (Anders 2012)).

With respect to the development of tertiary creep strains, it can be assumed in agreement to (Fernández Ruiz et al. 2007) that, at the moment of failure due to tertiary creep, approximately ⅔ of the total inelastic strains can be attributed to the secondary creep strains and ⅓ to the tertiary creep strains, see *Fig. 3-13*b. This is a hypothesis proposed by (Fernández Ruiz et al. 2007) based on their own experimental evidence as well as experimental evidence of (Maekawa and El-Kashif 2004). The applicability of this hypothesis is confirmed in section 3.6 of this paper. To evaluate the development of these tertiary creep strains, the following expression is proposed accounting for the ratio of the developed-to-available inelastic strains ($\varepsilon_{cc,in}/\varepsilon_{cc,in,av}$) and the level of stress:

$$\varepsilon_{cc,3}\left(t,t_0,\frac{\sigma_c}{f_c},\frac{\varepsilon_{cc,in}}{\varepsilon_{cc,in,av}}\right)=\gamma\left(\frac{\sigma_c}{f_c},\frac{\varepsilon_{cc,in}}{\varepsilon_{cc,in,av}}\right)\cdot\varepsilon_{cc,2}\left(t,t_0,\frac{\sigma_c}{f_c}\right) \tag{3.15}$$

For low values of $\varepsilon_{cc,in}/\varepsilon_{cc,in,av}$, the contribution of tertiary creep strains is negligible (almost no crack coalescence), whereas it increases at a growing rate (unstable crack coalescence) close to failure. On the basis of the experimental results of this paper (a comparison is presented in section 3.6.1), it is proposed to evaluate the value of the parameter γ according to:

$$\gamma = 0 \qquad \text{for } \sigma_c/f_c(t) < 0.75$$

$$\gamma = \frac{1}{2}\cdot\left(\frac{\varepsilon_{cc,in}}{\varepsilon_{cc,in,av}(t)}\right)^{\alpha} \qquad \text{for } \sigma_c/f_c(t) \geq 0.75; \tag{3.16}$$

where γ is only non-zero for stress levels above the assumed tertiary creep threshold of $\sigma_c/f_c = 0.75$, according to (Rüsch 1960). The parameter α governs the shape of the tertiary creep strain development and can reasonably be set equal to $\alpha = 4$. It can be noted that when $\varepsilon_{cc,in} = \varepsilon_{cc,in,av}$ (failure), it results $\varepsilon_{cc,3} = ½\varepsilon_{cc,2}$, which is consistent with the assumption that $\varepsilon_{cc,2} = ⅔\varepsilon_{cc,in,av}$ and $\varepsilon_{cc,3} = ⅓\varepsilon_{cc,in,av}$ (refer to *Fig. 3-13*b).

With respect to a variable loading history, the strain development can finally be calculated using the superposition principle as:

$$\begin{aligned}\varepsilon_c(t) = \sum_{i=1}^{n}\Bigg[&\varepsilon_{c0}(\sigma_{c,i})\left(1+\left[\eta\left(\frac{\sigma_{c,i}}{f_{c,i}}\right)+\gamma\left(\frac{\sigma_{c,i}}{f_{c,i}},\frac{\varepsilon_{cc,in,i}}{\varepsilon_{cc,in,av}}\right)\cdot\left(\eta\left(\frac{\sigma_{c,i}}{f_{c,i}}\right)-1\right)\right]\cdot\varphi(t,t_i)\right)\\ &-\varepsilon_{c0}(\sigma_{c,i-1})\left(1+\left[\eta\left(\frac{\sigma_{c,i-1}}{f_{c,i-1}}\right)+\gamma\left(\frac{\sigma_{c,i-1}}{f_{c,i-1}},\frac{\varepsilon_{cc,in,i-1}}{\varepsilon_{cc,in,av}}\right)\cdot\left(\eta\left(\frac{\sigma_{c,i-1}}{f_{c,i-1}}\right)-1\right)\right]\cdot\varphi(t,t_i)\right)\Bigg]\\ &+\left(\varepsilon_{cs}(t,t_s)-\varepsilon_{cs}(t_1,t_s)\right)\end{aligned} \tag{3.17}$$

This approach has been shown to be licit in combination with the affinity hypothesis of linear and nonlinear creep strains (Fernández Ruiz 2003) and considers the actual (nonlinear) instantaneous pre-peak strain for the applied level of stress (calculation of $\varepsilon_{c0}(\sigma_c)$ according to section 3.11). It can also be noted that this formula accounts for the concrete strength at the time of evaluation, and thus allows automatically to consider the increase of the concrete strength with time.

A comparison of this approach to the test results presented in this paper is discussed in the next section. It can be noted that this comparison is performed for standard cylinder dimensions (those described in section 3) typically used to characterize the compressive strength of concrete (a generalization to other potential geometries or sizes remains outside the scope of this work).

3.6 Comparison to test results

3.6.1 Own experimental programme

Eq. (3.17) was applied to predict the strain development of the experimental programme presented in section 3.4. On that basis, and by intersecting it with the failure criterion defined by the inelastic strain capacity of Eq. (3.18), the failure load can be calculated. The results are compared in *Fig. 3-14* for the 1st and 2nd test series (variable strain rate and stress rate respectively) as well as for the test series of Tasevski et al. (Tasevski et al. 2015, 2016).

The comparison shows a sound and consistent agreement for the various series both in terms of the strength and deformation capacity (detailed values presented in *Table 3-4*). In addition, the presented model suitably captures the observed experimental trends of decreasing strength and increasing deformation at failure for increasing time of application of the stress (*Fig. 3-14*). The results are also observed to be consistently reproduced for the two types of loading pattern investigated.

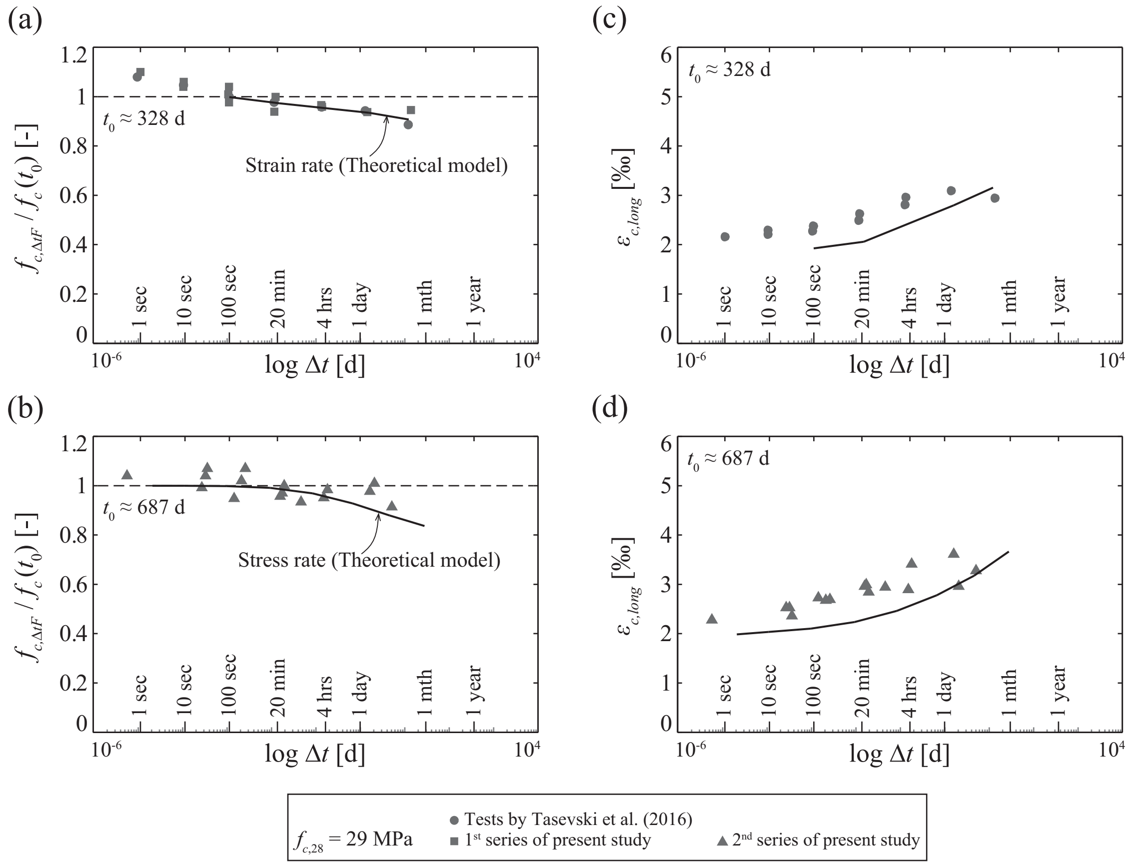

Fig. 3-14 Comparison of theoretical model and test results: (a) strength for strain rate tests; (b) strength for stress rate tests; (c) deformation at failure for strain rate tests; and (d) deformation at failure for stress rate tests

3.6.2 Tests with constant sustained stress

An additional validation of the proposed approach is performed by investigating tests from the research group of Rüsch (Rüsch 1960; Rüsch et al. 1968) and later re-evaluated by Grasser and Kraemer (Grasser and Kraemer 1985) on concrete specimens failing in compression under constant sustained stress. Also the experimental programmes by (Shank 1949; Awad and Hilsdorf 1971; Smadi et al. 1985; Iravani and MacGregor 1998) are included in the comparison. To estimate the shrinkage strains, a similar strategy has been applied as in section 3.4.1 (the Model Code 2010 formula has been adapted to best fit the shrinkage data provided in (Rüsch et al. 1968)). The effect of aging has been considered by using Eq.(3.6), where the coefficient s has been set equal to 0.25. Furthermore, to distinguish between linear and nonlinear creep strains, the primary creep development has been estimated from the curve at sustained stress level of $\sigma_c/f_c = 0.4$.

Table 3-4 Comparison of calculated failure stress and strain and test results

Test Name	$f_{c,\Delta t} / f_{c,ref}(t_0)$			$\varepsilon_{c,long}$		
	calc.	tested	tested / calc.	calc.	tested	tested / calc.
	[-]	[-]	[-]	[‰]	[‰]	[-]
DR4_2	0.974	0.999	1.026	2.05	2.62	1.28
DR5_1	0.953	0.966	1.014	2.41	2.81	1.17
DR5_2	0.953	0.959	1.006	2.41	2.95	1.22
DR6_1	0.935	0.939	1.004	2.78	3.09	1.11
DR7_1	0.906	0.932	1.029	3.18	2.94	0.925
		Avg	**1.016**		**Avg**	**1.14**
		StDev	**0.011**		**StDev**	**0.136**
		CoV	**0.011**		**CoV**	**0.119**
LR3_1	0.985	0.954	0.969	2.29	2.96	1.29
LR3_2	0.982	1.00	1.02	2.31	2.85	1.23
LR3_3	0.983	0.968	0.985	2.30	2.99	1.30
LR4_1	0.973	0.933	0.959	2.40	2.94	1.23
LR5_1	0.955	0.949	0.994	2.56	2.90	1.13
LR5_2	0.952	0.981	1.03	2.58	3.41	1.32
LR6_1	0.903	0.977	1.08	2.96	3.60	1.22
LR6_2	0.897	1.01	1.13	3.02	2.95	0.977
LR7_1	0.874	0.914	1.05	3.21	3.28	1.022
		Avg	**1.023**		**Avg**	**1.191**
		StDev	**0.055**		**StDev**	**0.123**
		CoV	**0.054**		**CoV**	**0.103**

The results are plotted in *Fig. 3-15* (Figures a-c comparing the strength and Figures d-f comparing the strain at failure). *Fig. 3-15* shows that a very good and consistent estimate is again obtained both in terms of strength and strain prediction. The model is able to capture the effect of long-term loading both on the decrease of strength as well as the increase of deformation capacity. This fine agreement between the tests and the prediction further confirms the applicability of the affinity hypothesis of nonlinear creep presented in section 4 of this paper (both for experiments on sustained loads from literature and for the own experimental programme on strain and stress rate tests).

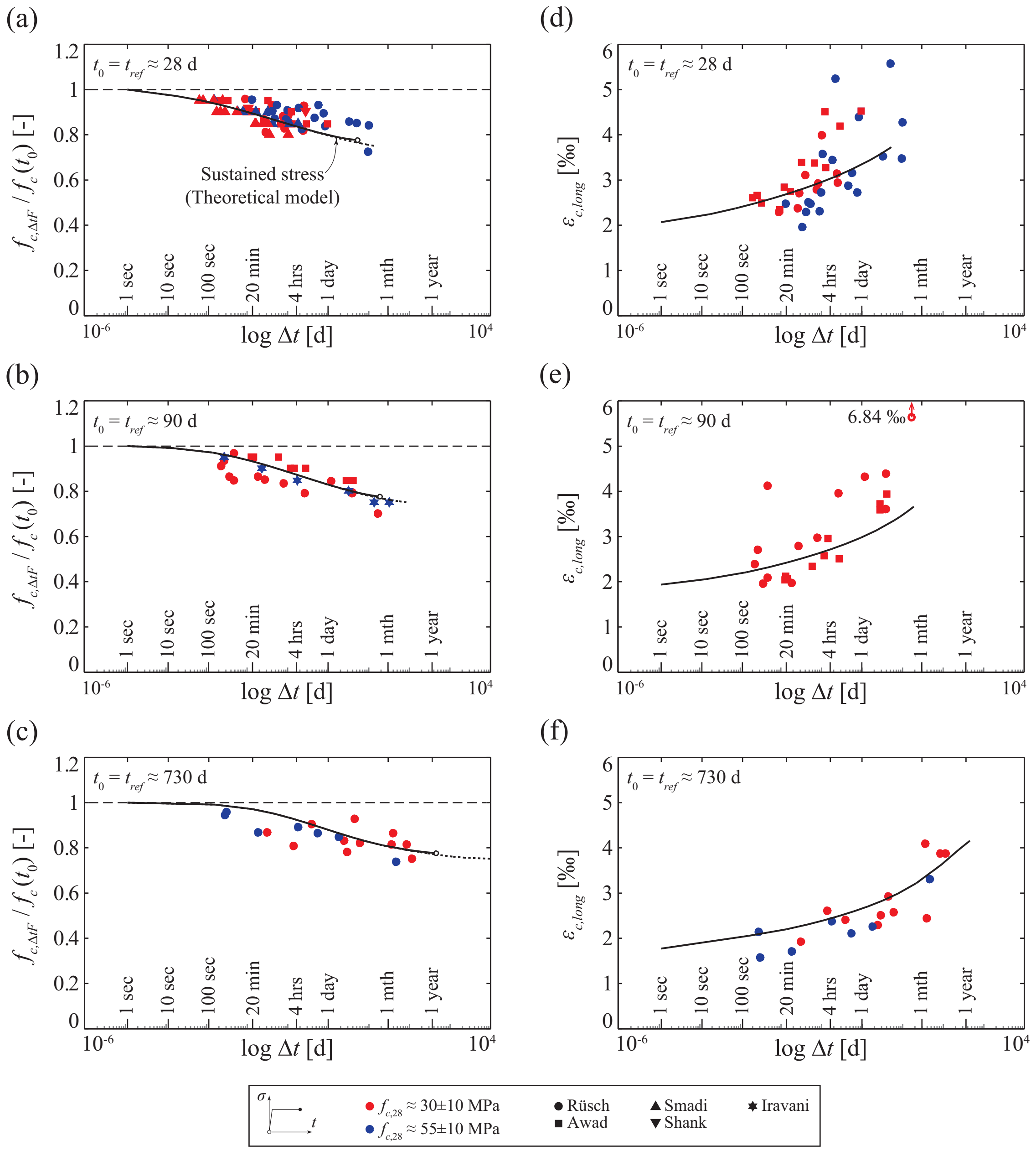

Fig. 3-15 Comparison of the proposed model to test results: (a-c) influence of sustained loading duration Δt on strength for different loading ages (28, 90 and 730 days respectively); and (d-f) influence on strain capacity. Solid line representing the analysis with increase of strength of concrete with time; dashed line representing the analysis without considering this effect. The model curves have been calculated for an average value $f_{c,28}$ = 40 MPa.

It can be noted that, in the plots of strength (*Fig. 3-15*a-c), two curves are provided. The dotted one corresponds to the analysis where failure is considered to occur without increase of concrete strength with time (no consideration of ageing influence), while the solid one accounts for this phenomenon. The main difference between both analyses can be seen in the fact that when the concrete ageing is considered (solid line), failures can develop only for smaller times and higher load levels than when this effect is not considered (dashed line). This can be seen in *Fig. 3-15*a-c by the circle indicated at the end of the solid line (considering concrete ageing), which corresponds to the last calculated point where failure under sustained loading can occur. After this point, the curve would theoretically raise (Rüsch 1960), although this part of the curve is physically not relevant with the described loading pattern. The results of the theoretical model also confirm the observations on this issue already stated by (Rüsch 1960) that failure is not prone to occur after a certain time when concrete is loaded at an early age (refer to *Fig. 3-15*a, the increase of strength with time is more significant than the progress of damage in the member). When the concrete is loaded at older ages, however, failures can occur after significant periods of time (refer to *Fig. 3-15*c).

The fact that the model is strain-based (and can thus predict the deformation capacity at failure) can additionally be considered as a significant quality. This allows understanding that, other than a detrimental effect on the strength of concrete, long-term actions also have a potential positive effect in redundant systems as the increased strains (more than proportionally) allows load to be redistributed from more stressed regions to less stressed regions of the structure (and for instance activate compression reinforcement).

3.7 Parametric analyses

Taking advantage of the model presented in section 3.5, some issues will be parametrically investigated in this section, namely the influence on the long-term response of concrete of the loading pattern, the ageing of concrete (increase of strength with time), the time at loading and the concrete strength.

3.7.1 Influence of loading pattern

The influence of the loading pattern on the concrete strength has been a matter of discussion since long. Rüsch (Rüsch 1960) already presented some considerations on the difference between the compressive strength of tests performed at constant strain rates and tests performed at sustained stress levels. His main conclusion was that "*constant loads lead to somewhat lower failure loads than loading at constant strain rates*". Based on the main assumptions made in section 3.5 of this paper, this (purely observational) conclusion of Rüsch can be confirmed from a theoretical point of view. To that aim, four loading patterns will be investigated: a constant sustained stress, a constant strain rate, a constant stress rate and a constant stress rate following a rapid loading ramp up to 80% of the material short-term strength (see *Fig. 3-16*).

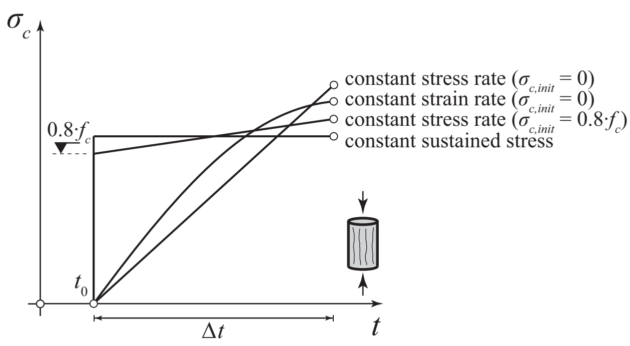

Fig. 3-16 Loading patterns investigated: constant sustained stress, constant strain rate, constant stress rate and constant stress rate applied after rapid loading up to 80% of material strength

The comparison of the response of concrete is presented in *Fig. 3-17* for a standard case ($f_{c,28}$ = 30 MPa, load applied two years after casting). The results show that failures under constant sustained stress occur earlier and for lower load levels than for the other loading patterns. This is in agreement with Rüsch's observations and is logical as concrete is subjected to higher stress levels since the beginning of the loading process (allowing for the propagation and coalescence of microcracks). For the same reason, from the three other cases, the constant stress rate starting at 80% of the material strength yields also to larger strength reductions than the constant strain and stress rates. Comparing the strain and stress rates, the strain rate, where the stress level remains fairly constant near failure, leads also (consistently to the other results) to larger strength reductions than the constant stress rate.

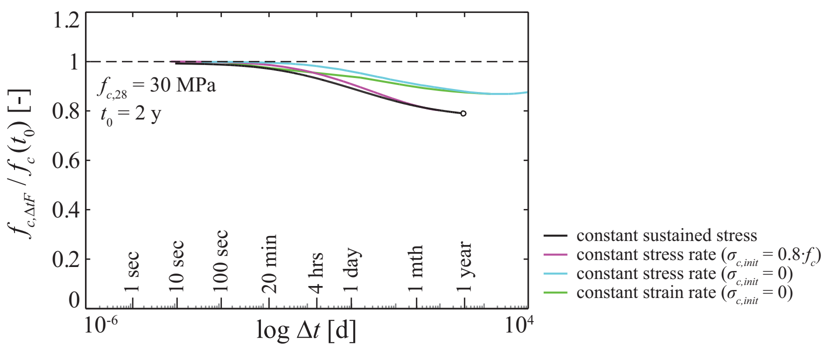

Fig. 3-17 Comparison of the calculated response for different types of loading: constant sustained stress, constant strain rate, constant stress rate and constant stress rate applied after rapid loading up to 80% of material strength

3.7.2 Influence of concrete ageing

Another important phenomenon that was already discussed in section 3.6.2 is the influence of concrete ageing on the long-term response of the material (increase of concrete strength with age, potentially compensating for the detrimental effect of the sustained loading). *Fig. 3-18* presents with this respect three plots prepared for identical concrete specimens but with different loading times and values of the parameter *s* (different strength classes of cement affecting the concrete ageing, refer to Eq. (3.6)). In the abscissa, the long-term strength is normalized with respect to the reference strength at 28 days in order to compare the absolute variations of strength.

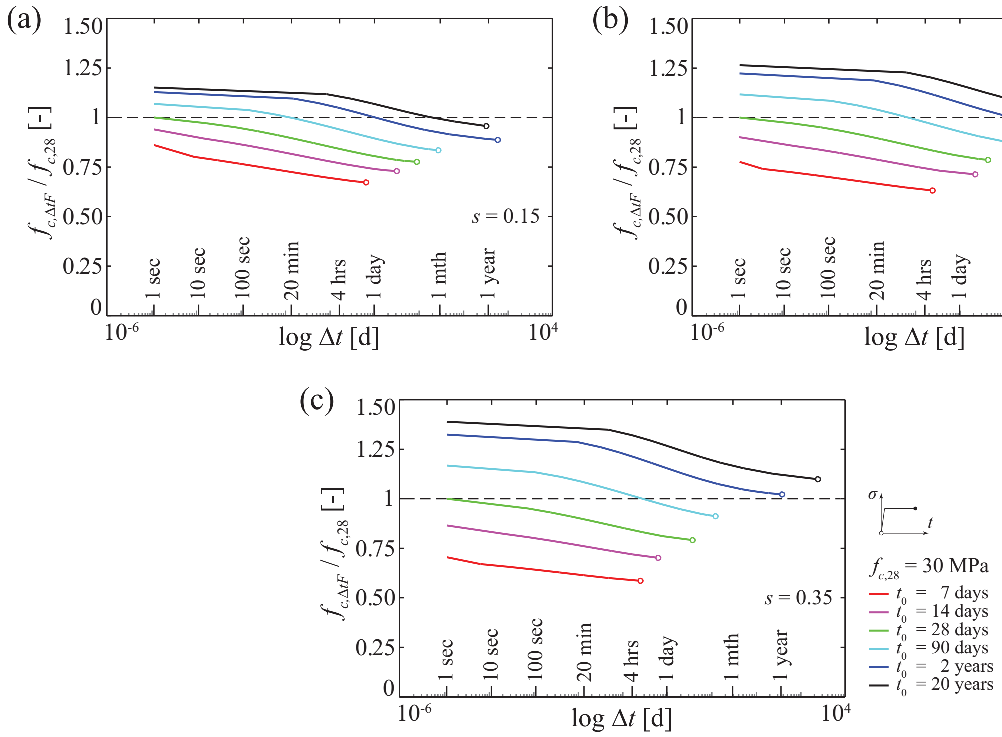

Fig. 3-18 Parametric analysis of the effect of the duration of sustained loading Δ*t* for different ages at initial loading ages t_0 (strength increase with concrete age considered normalized with the reference strength at 28 days): (a) *s* = 0.15; (b) *s* = 0.25 and (c) *s* = 0.35

As it can be seen in *Fig. 3-18*, when concrete is loaded at an early age, failure under sustained stress, if happens, occurs in a very limited period of time (1 day to 1 week). Thereafter, the increase of strength with time compensates for the material damage. For concrete loaded at older ages, however, failures can occur during longer periods of sustained stress (even more than one year) as little gain of the strength occurs. This result is consistent with Rüsch's conclusions (Rüsch 1960).

Additionally, it can be observed that when the concrete is loaded at old ages (black curves in *Fig. 3-18*), there is a large decrease of the strength under sustained loading. Yet, the long-term strength is normally similar or even higher to the one expected at 28 days (indicated by the value 1.0) as the concrete increased its strength prior to the loading process.

3.7.3 Influence of concrete strength and time at loading

Finally, some additional results are presented in *Fig. 3-19* with reference to the influence of concrete strength and time at loading. The influence of the concrete strength on the long-term loading (*Fig. 3-19*c) is not very sensitive. However, this parameter shows a very significant influence on the deformation capacity at failure (*Fig. 3-19*d). This is mostly justified by the fact that the maximum strength of higher concrete grades is developed for larger strain values.

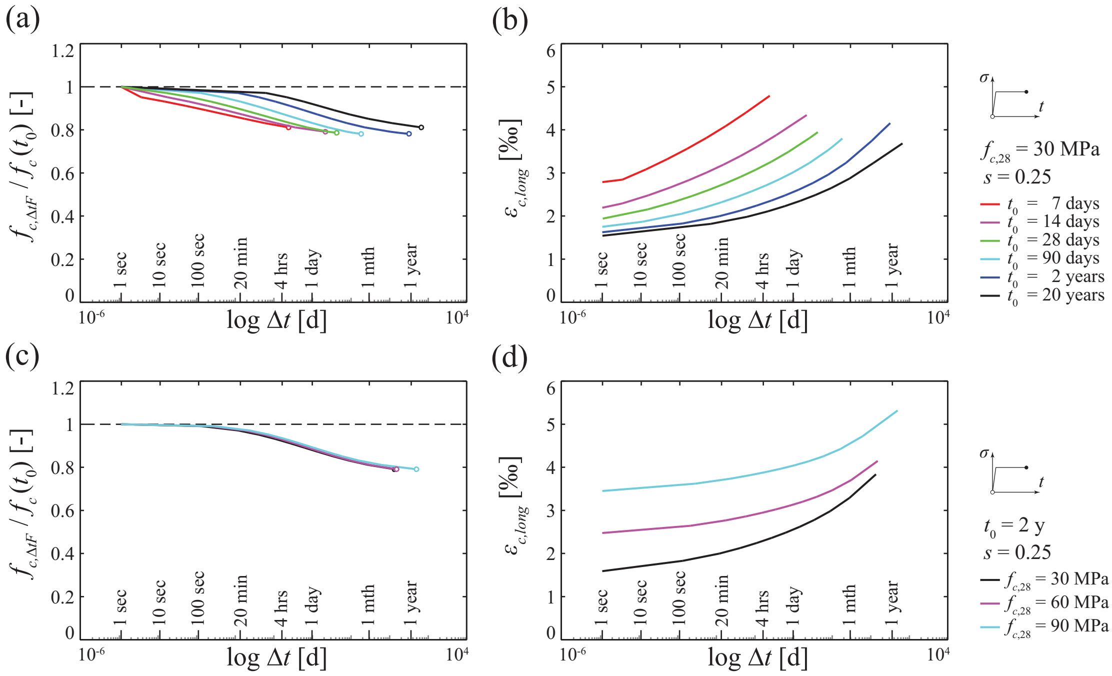

Fig. 3-19 Parametric analysis (strength increase with concrete age considered, strength normalized to the reference strength at the time of loading) of the effect of sustained loading Δt for different loading ages on: (a) strength; (b) strain at failure; and effect of concrete strength on: (c) strength; and (d) strain at failure

With respect to the strength decrease depending on the age at loading (*Fig. 3-19*a), this parameter was already investigated and discussed in section 3.7.2. Regarding the deformation capacity at failure (*Fig. 3-19*b), it can be seen that this parameter has a significant influence, with development of larger values of the deformations at failure for concrete members loaded at earlier ages. This result is justified by the fact that older concretes develop lower creep strains (linear and nonlinear) for the same duration of the load and are thus associated to lower deformation levels at failure.

3.8 Conclusions

This paper investigates on the development of linear and nonlinear creep strains potentially leading to failure under high levels of load. New experimental data is presented on loading patterns with variable strain and stress rates as well as an analytical approach allowing to consistently investigate these cases. The main conclusions of this paper are:

1. On the basis of the experimental results presented in this paper, failure under long-term load is shown to be governed by the inelastic strain capacity of concrete. When the developed inelastic strain of concrete equals the inelastic strain capacity, failure occurs by progressive coalescence of cracks. This result is confirmed for all the investigated loading patterns (constant sustained load, strain rate and stress rate).

2. The inelastic strain capacity can be estimated as the difference between the instantaneous post- and pre-peak strains for a given stress level. This allows defining a failure criterion for the available inelastic strain capacity.

3. The affinity assumption between linear and nonlinear creep strains (Fernández Ruiz et al. 2007) can be used as a simple and efficient tool to estimate the development of inelastic strains in concrete for a given loading history. The formulation of this approach has been extended in this paper:
    a. To be applicable to any potential loading pattern.
    b. To be applicable to failures at early age.
    c. To account in an explicit manner for the development of tertiary creep strains.

    The pertinence of this approach and new formulations has been validated with the experimental results presented in this manuscript.

4. Analytical calculation of failures under long-term loading patterns can be performed as the intersection between the relationship defining the development of inelastic strains for a given loading pattern (based on the affinity assumption) and the failure criterion defined by the inelastic strain capacity of concrete. This approach, confirmed by the experimental results presented in this paper as well as others from the scientific literature, allows to consistently estimate:
    a. The reduction of the strength due to sustained loading or low strain or stress rates. This is a detrimental effect in members subjected to high stresses.
    b. The increase of strains at failure. The evaluation of this effect, many times not explicitly acknowledged in design formulations, might nevertheless be an instrumental positive effect for statically redundant systems, where redistributions of stresses are possible from more stressed regions to less stressed regions and increasingly activate compression reinforcement.

5. On the basis of the theoretical model, some effects can be clearly reproduced and investigated, for instance:
    a. Constant sustained stress patterns lead to failures at lower stress levels and more rapidly than for stress or strain rates (concrete is subjected to higher stress levels since the beginning of the loading process).
    b. Ageing of the concrete (increase of concrete strength with time) is observed to be an instrumental phenomenon as it compensates for the material damage. For concretes loaded at early ages, failure can only occur after some hours or days, while for concretes loaded at older ages, failures under long-term load can occur several years after (as there is almost no increase of strength due to ageing).

3.9 Notation

Term	**Signification**
Variables	
E_c	modulus of elasticity of concrete
f_c	uniaxial compressive strength of concrete [MPa]
$f_{c,ref}(t_0)$	reference compressive strength obtained at a strain rate of 0.02 ‰·s^{-1} and at an age t_0 [MPa]
$f_{c,28}$	reference compressive strength obtained at an age of 28 days [MPa]
$f_{c,\Delta t}$	compressive stress at failure for a given long-term loading pattern [MPa]
t	time
t_0	concrete age at loading [days]
t_s	concrete age at the beginning of drying [days]
Δt	time under sustained stress for a constant stress level test or time after beginning of a strain or stress rate test (in the case of rapid initial loading, time after the initial loading ramp)
Δt_F	time under sustained stress or strain/stress rate required to attain failure
α	parameter for instantaneous stress–strain model
γ	parameter characterizing tertiary creep
$\dot{\varepsilon}$	strain rate
ε_c	concrete strain
ε_{c0}	instantaneous pre-peak strain
$\varepsilon_{c,trans}$	transverse strain
$\varepsilon_{c,long}$	longitudinal strain
φ	creep coefficient
η	affinity coefficient
ν	Poisson's ratio
$\dot{\sigma}$	stress rate
σ_c	concrete stress
Indexes	
av	available
c	concrete
calc	calculated
cc,1	primary creep
cc,2	secondary creep
cc,3	tertiary creep
cs	shrinkage
eff	effective
in	inelastic
init	initial

lin	linear
nl	nonlinear
test	value form experimental result
tot	total
τ	time-related

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3.11 Appendix 1: Instantaneous stress - strain curve

The instantaneous monotonic stress - strain curves used in this paper have been obtained based on the equation proposed in (Fernández Ruiz et al. 2007). This curve refers to the reference response of concrete, loaded in compression at a strain rate of $\dot{\varepsilon} = 0.02$ ‰·s^{-1} (approximately 100 seconds to failure) and is written as follows:

$$\sigma_c = \frac{E_c \cdot \varepsilon_c}{1 + \left(\frac{\varepsilon_c}{\varepsilon_{ref}}\right)^{\alpha}} \tag{3.18}$$

with

$$\varepsilon_{ref} = \frac{\alpha \cdot f_c}{E_c \cdot (\alpha - 1)^{\left(1 - \frac{1}{\alpha}\right)}} \tag{3.19}$$

and

$$\alpha = 0.5 + \frac{f_c[\text{MPa}]}{25} + \frac{(f_c[\text{MPa}])^2}{1500} \tag{3.20}$$

where the denominator in the second term of the α coefficient has been changed to 25 (original value was 20 (Fernández Ruiz et al. 2007)) in order to better approach the tested concrete behaviour (refer to *Fig. 3-20*).

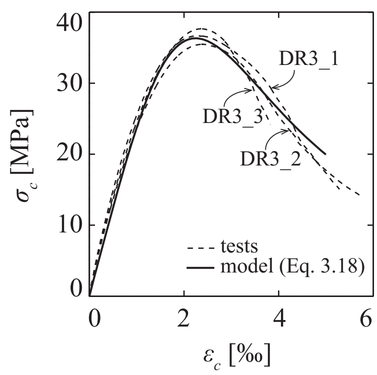

Fig. 3-20 Comparison between the instantaneous stress–strain curves from reference tests and the calculated one with Eq. (3.18)

Chapter 4 Assessing the compressive strength of concrete under sustained actions: from refined models to simple design expressions

This chapter presents the extension of the applicability of the theoretical framework of the previous chapter for any complex loading pattern involving high levels of sustained stress. The main aim of this chapter is to develop engineering design rules in form of simple code-like equations and to propose practical design recommendations. For this aim, the extended theoretical framework is explored for addressing typical design situations from the engineering practice, as for instance the time duration needed for the continued cement hydration to overcome the detrimental effect of sustained loads. Furthermore, the particular case of application of a variable action after a period of permanent load action is studied.

The chapter is a postprint version of a publication in a peer-reviewed academic journal. The authors are Darko Tasevski (PhD Candidate), Miguel Fernández Ruiz (Senior lecturer and thesis director) and Aurelio Muttoni (Professor and thesis director), and the full reference can be found here:

> **Tasevski, D., Fernández Ruiz, M. and Muttoni, A., (2019),** "Assessing the compressive strength of concrete under sustained actions: from refined models to simple design expressions." Structural Concrete, 2019, 1-15. (https://onlinelibrary.wiley.com/doi/full/10.1002/suco.201800303)

The main contributions of Darko Tasevski are the following:

- Extension of the analytical approach in collaboration with the thesis supervisors
- Numerical implementation of the extended analytical approach
- Performing all the calculations in the article
- Preparation of code-like equations (in collaboration with the second author)
- Production of the artwork in the article
- Preparation of the manuscript of the article

The second and third author (thesis directors) contributed to the understanding of the studied phenomena, proposing of theoretical concepts, interpreting of the analytical findings and writing of the final manuscript.

4.1 Abstract

The detrimental influence of high levels of sustained load on the compressive strength of concrete has been acknowledged and investigated since the 1950s. Despite the potential significance of this phenomenon in many situations, current design codes still provide limited guidance on how to account for this effect at ultimate limit state. In addition, the practical case of a significant permanent stress followed by stress increases due to variable loads is not addressed in most codes of practice. The aim of this paper is to present a general but simple design approach to account for the influence of sustained loading on the uniaxial behaviour of concrete in compression, both in terms of its strength and deformation capacity. This approach is based on a previously developed mechanical model accounting for nonlinear creep and material damage development on the long-term response of concrete. On that basis, a design approach is formulated in a simple code-like manner and used to investigate several practical design situations involving both sustained and variable actions. This approach is shown to be consistent and applicable to any general loading pattern, accurately reproducing the results of the general mechanical model and of available experimental results.

Keywords: deformation capacity, inelastic strain, internal redistribution, loading pattern, long-term strength, nonlinear creep, strain rate, sustained load

4.2 Introduction

As already well-established since the early developments of concrete, the compressive strength increases with time due to the continued cement hydration (Smeaton 1791) (see red curve in *Fig. 4-1*a). For this reason, already the first standards for reinforced concrete defined a concrete age (t_{ref}) to characterize the compressive strength of concrete and its associated mechanical properties (for instance 28 days according to the Swiss Code of 1903 (SIA 1903), 90 days according to the French Code of 1906 and 42 days according to the Austrian Code of 1907, refer to Mörsch (Mörsch 1912)). The reference time was in these codes in accordance to the rules related to the age at which removal of the scaffolding was allowed (which were usually lower than t_{ref}, see for instance the Swiss Code of 1903 (SIA 1903)). This approach already implicitly considered that additional permanent loads and variable actions are relevant for the structural performance in addition to self-weight, and that they are usually applied at a time larger than t_{ref}, see blue curve in *Fig. 4-1*a.

When assessing the structural performance at ultimate limit state, the uncertainties related to actions (higher stresses than expected) and the uncertainties related to the concrete strength (including the difference between the strength of the specimens for concrete testing and the local in-situ strength) have to be accounted for. This can be done by considering characteristic values for the actions and material properties and by applying pertinent partial safety factors (as performed in *Fig. 4-1*b with respect to *Fig. 4-1*a). This means that the verification is performed using design values for the internal actions and resistances. It can be noted that, in this frame of ultimate limit state analysis (which is a potential real situation, although with a very small probability of occurrence) the acting stresses may be relatively high compared to the material resistance (Kaltakci et al. 2007), which may be detrimental for the material strength (Hamed 2014).

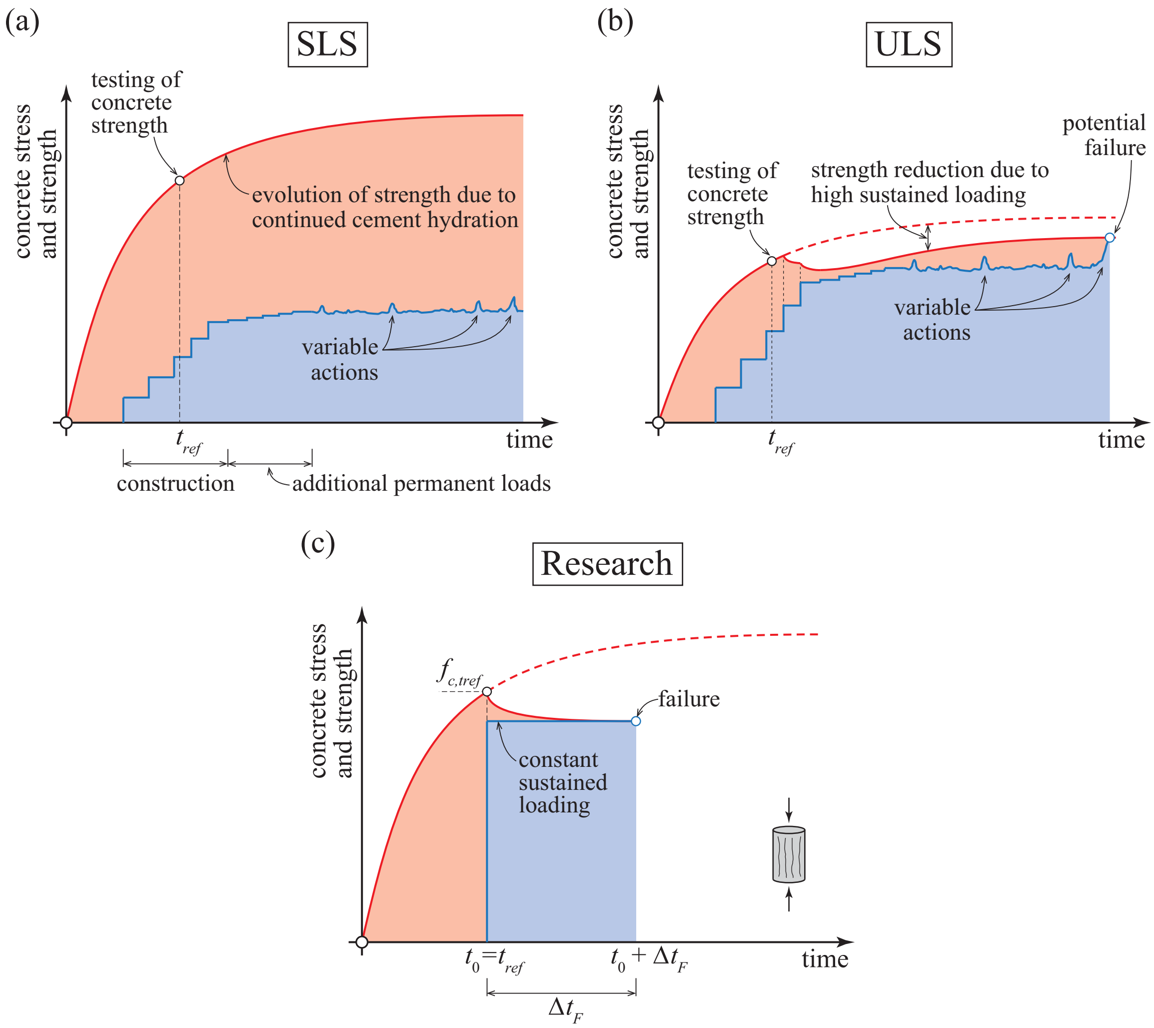

Fig. 4-1 Stress (blue) and strength (red) evolutions in time: (a) characteristic values for typical structures; (b) design values at Ultimate Limit State (ULS); and (c) typical tests investigating the effect of sustained load

This phenomenon (see *Fig. 4-1*b) was already observed in the 1950s by the pioneer works of Shank (Shank 1949) and later investigated in detail by Rüsch (Rüsch 1956, 1960; Rüsch et al. 1968). According to Rüsch (Rüsch 1960), when a high level of stress is applied to the concrete at a certain age (t_0), the strength of the material does not necessarily increase with time (as for unloaded specimens due to the continued cement hydration), but it may reduce. This occurs normally for stress levels above approximately 75%-80% of the strength of the material at the time of loading ($f_{c,t0}$, tested under standard conditions leading to failure in about 1-2 minutes according for instance to ISO 1920-4:2005 (ISO 2005)). This fact is sketched in *Fig. 4-1*c for a concrete specimen subjected to a constant level of high stress (but below the resistance of the concrete at the time of loading, $f_{c,t0}$), showing that failure occurs after some time of application of the action. According to these observations, Rüsch (Rüsch 1960) established the concept of sustained load strength, that he defined as the maximum stress level at which concrete can be permanently loaded without failure.

For the case of actual structures, the detrimental influence of high levels of sustained load is only notorious after a significant fraction of the permanent load has been applied to the structure. This can be seen in *Fig. 4-1*b, by the decay in the red curve (material resistance) for high levels of permanent load. Thereafter, the strength may increase again due to the continued cement hydration, but in any case the strength is reduced with respect to that of an unloaded material (dashed red curve in *Fig. 4-1*b). Failure will eventually occur when the action equals the resistance, which may take place at different moments depending on the acting variable actions and material decay on the strength (as already observed by Rüsch (Rüsch 1960) and others (Stöckl 1972; fib 2013a; Tasevski et al. 2018), see *Fig. 4-1*b).

Due to its practical relevance, the topic of the long-term strength of concrete structures has attracted many research efforts after the works of Rüsch, with a number of investigations conducted both for normal strength concrete (Shah and Chandra 1970; Awad and Hilsdorf 1971; Diaz and Hilsdorf 1971; Wittmann and Zaitsev 1972; Stöckl 1972; Coutinho 1977; Fouré 1985; Smadi et al. 1985; Fernández Ruiz et al. 2007; Schlappal et al. 2017; Tasevski et al. 2018) and high strength concrete (Ngab et al. 1981; Smadi 1983; Smadi et al. 1985; Han and Walraven 1994; Iravani and MacGregor 1998; Müller et al. 2010). The same phenomenon has been also investigated for the tensile and flexural behaviour under sustained load (see for instance references (Domone 1974; Reinhardt and Cornelissen 1985; Reinhardt and Rinder 2006)). Within this frame, most experimental evidence with respect to the effect of sustained loading on the compressive strength has been obtained by considering a constant stress level applied to the member (refer to the load pattern shown in *Fig. 4-1*c). However, some experimental programmes and researches have also reported this phenomenon for low loading rates (El-Kashif and Maekawa 2004; Fernández Ruiz et al. 2007; Fischer et al. 2014). Several authors have also investigated the origin of the phenomenon, relating the delayed failure of concrete to microcracking and to material damage development and progression (Wittmann and Zaitsev 1972; Berthollet et al. 2004; Fernández Ruiz et al. 2007; Rossi et al. 2013). According to these research works, if the level of sustained stress is higher than the threshold at which microcrack propagation is likely to occur ($\sigma_c/f_c > 0.4$, typically associated to the development of nonlinear creep strains), delayed damage may develop in the material. At elevated sustained stress levels ($\sigma_c/f_c > 0.75$, corresponding to Rüsch's sustained load strength), a significant amount of delayed damage may develop, yielding to unstable crack propagation over time and potentially to material failure.

For moderate levels of permanent load (σ_c between approximately $0.4{\cdot}f_c$ and $0.6{\cdot}f_c$) applied to the material, a small beneficial influence of sustained levels of stress has been observed on the material strength(Wittmann and Zaitsev 1972). This fact, confirmed experimentally (Wittmann and Zaitsev 1974), has been attributed to the beneficial internal stress redistributions occurring in the material (Wittmann and Zaitsev 1972) (reducing stress concentrations; alternative explanations of this phenomenon can be consulted elsewhere (Claisse and Dean 2013)), but is usually neglected in practice.

With respect to codes, the approach for the design of new structures usually considers that the detrimental effect of sustained loads on the concrete strength can be compensated by the increase of strength due to continued cement hydration after the reference time t_{ref}. This is for instance the approach followed by MC2010 (fib 2013b) and EN 1992-1-1:2004 (CEN 2004) when the reference strength is considered at a time t_{ref} equal to 28 days. However, when the reference time for evaluation of the compressive strength is higher (t_{ref} > 28 days), the uniaxial compressive strength has to be reduced accordingly to account for the effects of sustained loading (refer for instance to coefficient k_t in EN 1992-1-1:2004 (CEN 2004), recommended to be 0.85 in §3.1.2(4)). This is a typical situation for the assessment of existing structures, where the concrete strength can be updated after long periods of service of the structure and almost no strength increase due to cement hydration can be expected anymore (this also presumes that concrete has suffered no degradation processes such as alkali-aggregate reaction, sulphate attack or freeze-thaw cycles). However, when a general loading case is considered (refer for instance to *Fig. 4-1*b) there is usually little or no guidance on how to determine in a simple and consistent manner the uniaxial compressive strength of concrete.

In this context, the aim of this paper is to provide a simple analytical approach to evaluate the compressive strength of concrete when subjected to long-term actions and accounting for potentially high levels of stress. The topic is analysed by using the theoretical frame of Tasevski et al. (Tasevski et al. 2018), which provides a model to assess the material damage and its influence both in terms of strength and deformation capacity. The model is consistently verified with test results considering different loading patterns (sustained stress, stress rates and strain rates) and formulated in a design-friendly manner. On that basis, several practical design situations are analysed, deriving a number of practical recommendations.

4.3 Concrete strength in case of constant sustained loading

A number of approaches on the modelling of nonlinear creep and associated damage as well as for prediction of delayed failure of concrete can be found in literature. Many of these approaches are based on material damage development (Hellesland and Green 1972; Carol and Murcia 1989; Mazzotti and Savoia 2003; Bockhold and Stangenberg 2004; El-Kashif and Maekawa 2004; Challamel et al. 2005; Fernández Ruiz et al. 2007; Tasevski et al. 2018) and others are based on fracture mechanics (Wittmann and Zaitsev 1972; Zhou 1992; van Zijl et al. 2001; Barpi and Valente 2002; Di Luzio 2009). Within this work, the approach of Tasevski et al. (Tasevski et al. 2018) will be followed. This approach is based on the hypothesis of affinity between linear and nonlinear creep strains (proposed by Fernández Ruiz et al. (Fernández Ruiz et al. 2007)) and accounts for material damage development associated to nonlinear creep. This chapter gives a short review of this approach.

4.3.1 Nonlinear creep strains

If concrete is subjected to a compressive stress lower than ~$0.4 \cdot f_c$, the correlation between the delayed creep strain and the instantaneous strain is almost linear (*Fig. 4-2*a) and thus governed by a linear creep phenomenon without development of any significant material damage. At higher stress levels ($\sigma_c/f_c > 0.4$), the linear correlation between the delayed creep strain and the instantaneous strain is lost and additional nonlinear creep strains develop (Tasevski et al. 2018). This loss of linear correlation is due to the damage process in the material associated to concrete microcracking.

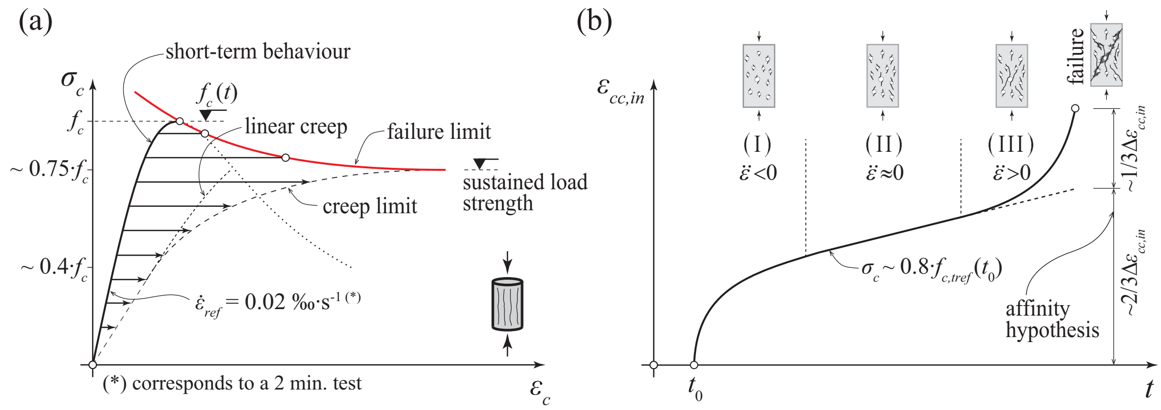

Fig. 4-2 Uniaxial compressive behaviour of concrete: (a) short- and long-term stress-strain response, adapted from Rüsch (Rüsch 1960); (b) stages of creep: primary, secondary and tertiary; definition of the affinity hypothesis

According to several authors (Zhou 1992; Berthollet et al. 2004; Bockhold and Stangenberg 2004), the development of inelastic strains associated to material damage can be separated into three phases (*Fig. 4-2*b). In the primary creep phase (immediately after load application), the strains develop at a high initial rate and stabilize with time. In the following (secondary creep) phase, the rate is fairly constant. At high load levels, the microcrack propagation becomes unstable and leads to failure of the specimen (see *Fig. 4-2*), corresponding to the tertiary creep phase. This latter phase is characterized by an increasing rate of strains, resulting in an uncontrolled process of crack coalescence.

4.3.2 Failure criterion

Following the approach of Fernández Ruiz et al. (Fernández Ruiz et al. 2007), the inelastic strain capacity $\varepsilon_{cc,in,av}$ for a given stress level defined as the difference between the instantaneous post- and pre-peak strains for that level of stress (*Fig. 4-3*a) can be assumed as the governing parameter for the failure criterion. This value can be directly calculated by using the monotonic stress-strain curve of concrete in compression, as further verified experimentally by Tasevski et al. (Tasevski et al. 2018). The monotonic response is considered to be obtained at a reference strain rate of $\dot{\varepsilon}$ = 0.02 ‰·s^{-1} (approximately 2 minutes to reach the maximum strength). Failure is considered to occur when the total inelastic strain developed by the material (accounting for secondary and tertiary creep strains) equals the inelastic deformation capacity (see *Fig. 4-3*b).

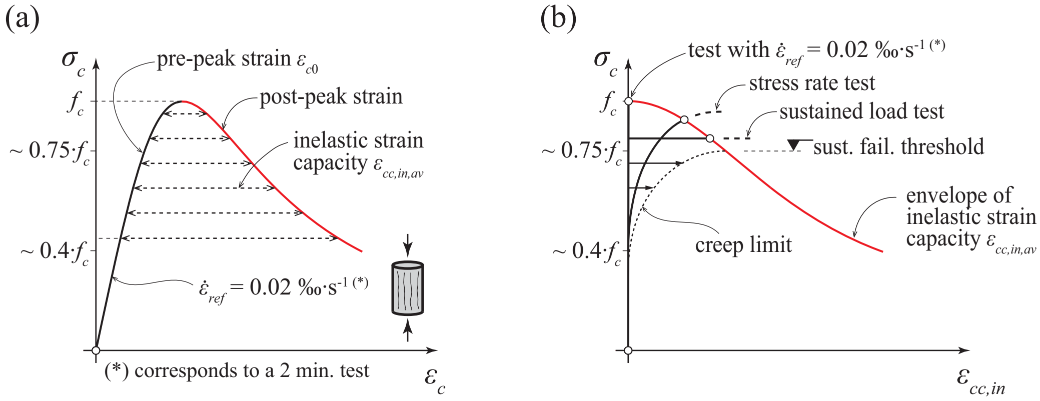

Fig. 4-3 Inelastic strain capacity $\varepsilon_{cc,in,av}$ according to Fernández Ruiz et al.(Fernández Ruiz et al. 2007): (a) definition of $\varepsilon_{cc,in,av}$ as the difference between post- and pre-peak strains of the monotonic curve; (b) definition of failure as intersection of the stress – inelastic strain curve and the failure envelope of $\varepsilon_{cc,in,av}$ (Tasevski et al. (Tasevski et al. 2018))

Similar approaches by other authors have been reported, where the monotonic stress-strain curve is used as a failure criterion for fatigue life (Karsan and Jirsa 1969; Zanuy et al. 2010; Hamed 2014). Details on the development of inelastic strain related to material damage and on the calculation of failure can be found in Tasevski et al. (Tasevski et al. 2018) and are summarized in the following section.

4.3.3 Development of inelastic strain by concrete at high stress levels

According to Tasevski et al. (Tasevski et al. 2018), the strains developed for a concrete loaded at time t_0 can be described in a general manner by the following expression:

$$\varepsilon_c\left(t,\frac{\sigma_c}{f_c}\right) = \varepsilon_{c0}\left(t_0,\frac{\sigma_c}{f_c}\right) + \Delta\varepsilon_{cs}(t,t_0) + \Delta\varepsilon_{cc}\left(t,t_0,\frac{\sigma_c}{f_c}\right) \tag{4.1}$$

where the first term on the right side of the equation ($\varepsilon_{c0}\left(t_0,\frac{\sigma_c}{f_c}\right)$) corresponds to the instantaneous pre-peak strain (including elastic and inelastic components, refer to the short-term behaviour in *Fig. 4-2*a), the second ($\Delta\varepsilon_{cs}(t,t_0)$) to the shrinkage strains (assumed not to be associated to any material damage) and the third ($\Delta\varepsilon_{cc}\left(t,t_0,\frac{\sigma_c}{f_c}\right)$) to the creep strains (Tasevski et al. 2018).

The creep strains can be divided into primary, secondary and tertiary creep in the following manner:

$$\varepsilon_{cc}\left(t,t_0,\frac{\sigma_c}{f_c}\right) = \varepsilon_{cc,1}(t,t_0) + \varepsilon_{cc,2}\left(t,t_0,\frac{\sigma_c}{f_c}\right) + \varepsilon_{cc,3}\left(t,t_0,\frac{\sigma_c}{f_c},\frac{\varepsilon_{in}}{\varepsilon_{in,av}}\right) \tag{4.2}$$

where the primary creep strains are calculated by means of the linear creep coefficient (and are not associated to material damage):

$$\varepsilon_{cc,1}(t,t_0) = \varphi_{lin}\cdot\varepsilon_{c0} \tag{4.3}$$

The secondary creep strains (due to micro-cracking development and thus associated to material damage) are evaluated as (Fernández Ruiz et al. 2007):

$$\varepsilon_{cc,2}\left(t, t_0, \frac{\sigma_c}{f_c}\right) = (\eta - 1) \cdot \varepsilon_{cc,1}(t, t_0) \tag{4.4}$$

where the coefficient η is expressed as follows (Fernández Ruiz et al. 2007; Tasevski et al. 2018):

$$\eta\left(\frac{\sigma_c}{f_c}, t, t_0\right) = 1 + 2 \cdot \eta_\tau(t, t_0)\left(\frac{\sigma_c}{f_c(t)}\right)^4 \tag{4.5}$$

and coefficient η_τ takes into account the development of the nonlinear creep strains with time (Tasevski et al. 2018):

$$\eta_\tau(t, t_0) = \left(1 - \log\left(\frac{t - t_0}{t_m + t - t_0}\right)\right)^n \tag{4.6}$$

where constant values t_m = 100 days and n = 0.75 can be generally assumed (Tasevski et al. 2018). It can be noted that when $t \rightarrow \infty$, then $\eta_\tau \rightarrow 1$ and consequently Eq. (4.5) takes the form originally suggested by Fernández Ruiz et al. (Fernández Ruiz et al. 2007) for nonlinear creep strains after a long time.

The development of tertiary creep strains (associated to micro-crack coalescence and thus also to material damage) can be evaluated by taking into accounting the ratio of developed-to-available inelastic strains ($\varepsilon_{cc,in}/\varepsilon_{cc,in,av}$) and the level of stress, calculated by means of the following equation (Tasevski et al. 2018):

$$\varepsilon_{cc,3}\left(t, t_0, \frac{\sigma_c}{f_c}, \frac{\varepsilon_{cc,in}}{\varepsilon_{cc,in,av}}\right) = \gamma\left(\frac{\sigma_c}{f_c}, \frac{\varepsilon_{cc,in}}{\varepsilon_{cc,in,av}}\right) \cdot \varepsilon_{cc,2}\left(t, t_0, \frac{\sigma_c}{f_c}\right) \tag{4.7}$$

In the ratio $\varepsilon_{cc,in}/\varepsilon_{cc,in,av}$, the term $\varepsilon_{cc,in,av}$ corresponds to the total available inelastic strain and is characterised by the short-term monotonic response of concrete (*Fig. 4-3*a). The term $\varepsilon_{cc,in}$ corresponds to the developed nonlinear creep strain (total strain minus shrinkage, instantaneous and linear creep strains).

For low values of the ratio $\varepsilon_{cc,in}/\varepsilon_{cc,in,av}$, tertiary creep strains have a negligible contribution, whereas they increase at a growing rate when failure approaches. As suggested by Tasevski et al. (Tasevski et al. 2018), the parameter γ can be calculated with help of the following expression:

$$\gamma = \frac{1}{2} \cdot \left(\frac{\varepsilon_{cc,in}}{\varepsilon_{cc,in,av}(t)}\right)^{\alpha} \qquad \text{for } \sigma_c/f_c(t) \geq 0.75 \tag{4.8}$$

According to this assumption, γ is zero for stress levels below the threshold of possible tertiary creep development (σ_c/f_c = 0.75). The shape of the tertiary creep curve is governed by the parameter α, which can be considered equal to 4, based on the analysis of experimental results by Tasevski et al. (Tasevski et al. 2018). At failure, $\varepsilon_{cc,in} = \varepsilon_{cc,in,av}$ which results $\varepsilon_{cc,3} = ½\varepsilon_{cc,2}$. This formula is in addition consistent with the affinity hypothesis originally formulated by Fernández Ruiz et al. (Fernández Ruiz et al. 2007) that considers $\varepsilon_{cc,2} = \frac{2}{3}\varepsilon_{cc,in,av}$ and $\varepsilon_{cc,3} = \frac{1}{3}\varepsilon_{cc,in,av}$ (*Fig. 4-2*b).

A significant advantage of the presented model is the fact that it is strain-based and allows thus tracking the amount of material damage for any loading history, and to calculate both the strength and deformation capacity of concrete for any potential loading pattern.

4.3.4 Comparison of the developed model to the *fib* MC2010 approach and proposal of an analytical expression

According to *fib* MC2010 (fib 2013b) (Eq. (5.1-53)), the compressive concrete strength in case of a constant sustained loading starting at time t_0 and for a duration Δt_F can be expressed by (Eq. (5.1-53) of *fib* MC2010 (fib 2013b), notation adapted consistently to the one of this paper):

$$f_c(t_0 + \Delta t_F) = f_{c,28} \cdot \beta_{cc}(t_0 + \Delta t_F) \cdot \beta_{c,sus}(\Delta t_F) \tag{4.9}$$

In this expression, the influence of two effects is considered: the increase of strength due to continued cement hydration ($\beta_{cc}(t_0+\Delta t_F)$) which depends on the age at failure $t_0+\Delta t_F$ and the strength reduction due to sustained loading ($\beta_{c,sus}(\Delta t_F)$) which depends on the load duration Δt_F. The parameter $\beta_{cc}(t_0+\Delta t_F)$ can be calculated for t_{ref} = 28 days with Eq. (5.1-51) of *fib* MC2010 (fib 2013b) (notation adapted consistently to the one of this paper):

$$\beta_{cc}(t_0 + \Delta t_F) = e^{s \cdot \left(1 - \sqrt{\frac{28}{t_0 + \Delta t_F}}\right)} \tag{4.10}$$

where s is a coefficient that depends on the cement type. For t_{ref} different to 28 days, Eq. (4.10) has to be replaced by:

$$\beta_{cc}(t_0 + \Delta t_F) = e^{s \cdot \left(1 - \sqrt{\frac{t_{ref}}{t_0 + \Delta t_F}}\right) \cdot \sqrt{\frac{28}{t_{ref}}}} \tag{4.11}$$

With respect to the parameter $\beta_{c,sus}(\Delta t_F)$ in Eq. (4.9), it accounts for the strength reduction due to sustained loading, and is evaluated by means of the following expression (Eq. (5.1-54) from *fib* MC2010 (fib 2013b), notation adapted consistently to the one of this paper):

$$\beta_{c,sus}(\Delta t_F) = 0.96 - 0.12 \cdot \left(\ln(72 \cdot \Delta t_F)\right)^{1/4} \tag{4.12}$$

where Δt_F is measured in days. This expression has been obtained as a best fit of test results by the research group of Rüsch (Rüsch 1960; Rüsch et al. 1968). It can be noted that, since the evaluation of the test data was made adding the initial loading time (20 minutes for those tests) to the sustained loading duration, the expression is in principle only applicable for $\Delta t_F > 0.015$ days (= 20 minutes). In addition, the coefficient 0.96 in this expression covers the difference of a test loaded in 20 minutes and a standard test loaded in approximately 2 minutes (CEB 1962).

Fig. 4-4 presents a comparison of these expressions with the model described in the previous section and the test data gathered in Tasevski et al. (Tasevski et al. 2018) collected from several research groups (Shank 1949; Rüsch et al. 1968; Awad and Hilsdorf 1971; Smadi et al. 1985; Iravani and MacGregor 1998). The sustained loading duration has been corrected to account consistently for the duration of the initial loading phase. It can be seen in that figure that the *fib* MC2010 (fib 2013b) expressions (green lines in *Fig. 4-4*a-c) provide a lower limit of the measured strength under sustained loading. Furthermore, the formula is only valid for times of application of the load larger than 20 minutes, which makes the expression rather sensitive (strong drop on the strength after 20 minutes) and limits its field of application. In addition, the coefficient $\beta_{c,sus}(\Delta t)$ according to MC2010 does not depend on the concrete age at loading (t_0), but only on the time duration for which the load is applied (Δt). However, according to the test results and the theoretical model (Tasevski et al. 2018), the age of concrete at loading is a significant parameter.

Fig. 4-4 also plots the results of the model by Tasevski et al. (Tasevski et al. 2018) (black solid curves in *Fig. 4-4*). This approach yields to a smooth and gradual reduction of the strength with increasing times of application of the load.

Accounting for these facts, it is considered that current *fib*'s MC2010 formula can be improved to yield to more consistent predictions of the behaviour and to expand its range of validity to short-time applications of the load (lower than 20 minutes). On the basis of the results presented in this paper and by analysis of the results, the following analytical expression is proposed for the phenomenon of strength reduction due to high levels of sustained loading:

$$\beta_{c,sus}(t_0, \Delta t_F) = \lambda(t_0) + \frac{1 - \lambda(t_0)}{\left(1 + k_2 \frac{\Delta t_F}{t_0}\right)^{1/k_1}} \tag{4.13}$$

This expression is derived by fitting of the numerical predictions of the general model by Tasevski et al. (Tasevski et al. 2018) with a simple analytical equation depending both on t_0 and Δt_F. It can be noted that Eq. (4.13), differently to Eq. (4.12), shows an explicit dependence on t_0 as: (1) the inelastic strain development is based on a creep model that depends on t_0 and Δt_F, and a monotonic response that depends on t_0; (2) the failure envelope depends on the monotonic response which is calculated at $t_0+\Delta t_F$.

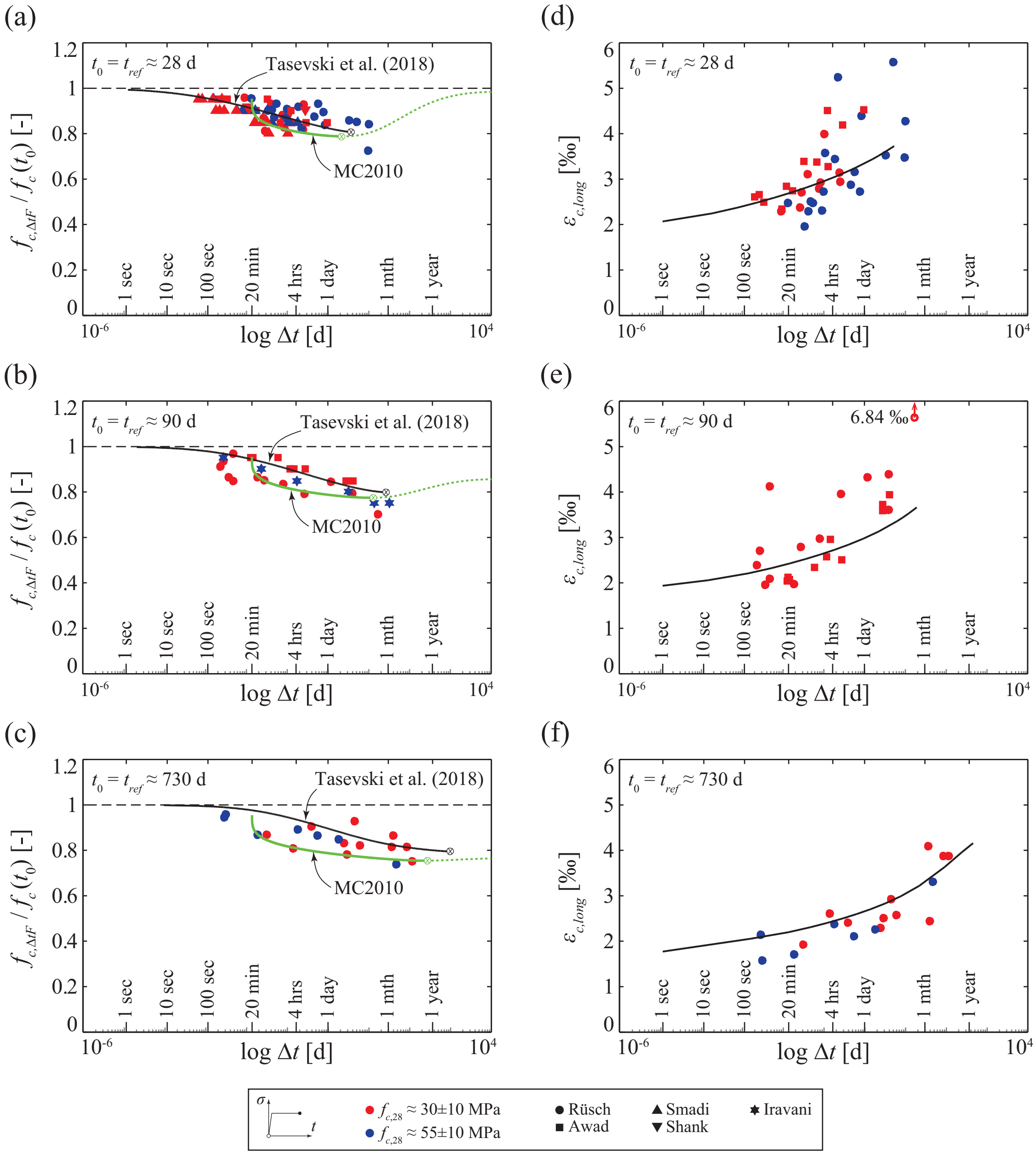

Fig. 4-4 Comparison of experimental data points with the theoretical model (Tasevski et al. 2018) (solid black lines, Tasevski et al. (Tasevski et al. 2018)) and *fib* MC2010 (Eq. (4.12)), solid green lines) for constant sustained stress (s = 0.25)

With respect to the parameters of Eq. (4.13), the values $k_1 = 10$; $k_2 = 10^4$ and $\lambda(t_0) = 0.64 + 0.01 \cdot \ln(t_0)$ (with t_0 in days) can be adopted yielding to overall accurate and consistent results. A comparison of the results of the model by Tasevski et al. (Tasevski et al. 2018) and those of Eq. (4.13) can be seen in *Fig. 4-5*. A nice agreement is found, yielding in addition to a good approximation of the sustained load strength (values about 80% of the reference strength, indicated with a circle in *Fig. 4-4*). It shall be noted that *Fig. 4-5* presents the failure of concrete under constant levels of stress. Cases where the material does not fail and additional loading can be applied will be discussed in section 4.4.

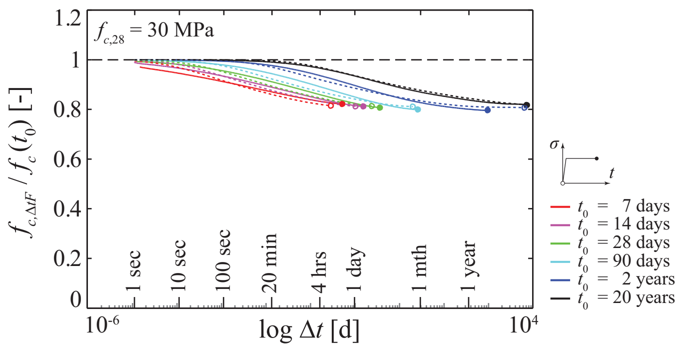

Fig. 4-5 Comparison of theoretical model (Tasevski et al. 2018) (solid lines) and analytical Eq. (4.13) (dashed lines) for a constant sustained stress ($f_{c,28}$ = 30 MPa and s = 0.25)

4.3.5 Deformation capacity and redistribution of internal forces

Due to the fact that the development of nonlinear creep strains at high stress levels may potentially lead to failure under sustained load, this phenomenon has traditionally been considered as a detrimental effect for concrete. With this respect, it shall nevertheless be noted that the development of nonlinear creep increases the deformations of the material at a non-proportional rate with respect to regions where linear creep governs. This phenomenon can be observed in *Fig. 4-2*a, where the curve defining the creep limit (deformations of the material accounting for instantaneous and creep strains) loses the proportionality for stress levels above 40% of the (short-term) compressive strength of the material. Some test results as well as the predictions of the theoretical model by Tasevski et al. (Tasevski et al. 2018) with respect to this phenomenon can also be seen in *Fig. 4-4*d-f, where the deformation capacity at peak load is plotted as a function of the load duration.

As a consequence of this nonlinear response in terms of strains, redistributions of stresses can occur between regions deforming more (those subjected to high level of stresses and developing nonlinear creep strains) and those deforming less (those subjected to a linear creep response). This is a potentially beneficial influence of the development of nonlinear creep strains as it contributes reducing the stress level at the most stressed regions.

One clear example of this beneficial influence can be observed in the compression zone of members in bending, where the outermost compressed fibres reduce relatively their stress due to nonlinear creep. As experimentally observed (Rüsch 1960; Mattock et al. 1961), this phenomenon justifies why adopting a parabolic/constant stress distribution in the compression zone due to bending is more realistic than adopting one based on the short-term material response and considering a linear profile of strains (Fernández Ruiz 2003), see *Fig. 4-6*.

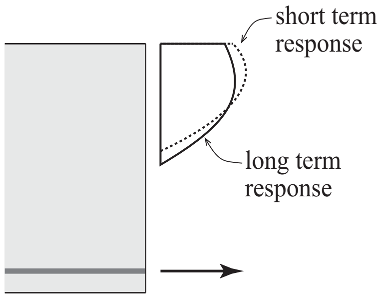

Fig. 4-6 Compression zone of a member in bending with stress distribution under short-term and long-term loading

Another example where this phenomenon is significant refers to the response of reinforced concrete columns with limited 2nd order effects. In these cases, governed by the compressive strength of concrete and by the presence of the longitudinal reinforcement steel, the additional deformation due to creep allows to increase the contribution of the compression reinforcement (which is potentially not yielded when concrete crushes under rapid loading conditions). Thus, while slow loading rates or sustained loading may be detrimental for the strength of columns with low longitudinal reinforcement ratios (low contribution of the reinforcement to the overall strength), it may be beneficial for columns with large reinforcement ratios (additional contribution of the reinforcement compensating for the decrease of the concrete contribution).

4.4 Concrete strength for different loading patterns

4.4.1 Introduction

Despite the fact that most research on the topic of nonlinear creep behaviour has been performed on specimens loaded under a constant sustained stress (*Fig. 4-1*c), structures are seldom subjected to such type of loading. On the contrary, refer for instance *Fig. 4-1*b, most structures are subjected to other loading patterns. With this respect, three loading patterns can be identified as particularly relevant for practice: approximately monotonic stress rates (*Fig. 4-7*a, typically referring to the construction sequence); approximately monotonic stress rates after application of a first loading ramp (*Fig. 4-7*b, typically referring to the application of dead loads after construction) or loading patterns with a rapid increase of load after a period of quasi-sustained loading (*Fig. 4-7*c, typically referring to live loads in addition to permanent loads).

With reference to these patterns, design codes usually provide no general method to assess their structural response accounting for the progression of material damage with time. In addition, refined and general approaches (as the one of Tasevski et al. (Tasevski et al. 2018)) may be too complex for their application in practice. In the following, a simple methodology to calculate the response of a structure subjected to a general loading pattern will thus be presented. The approach is based on the Palmgren-Miner's rule, whose pertinence will be justified from a theoretical perspective and by comparison to test results. By making use of this tool, the various loading patterns relevant for practice shown in *Fig. 4-7* will be investigated (cases (a-b) in section 4.4.3 and case (c) in section 4.4.4).

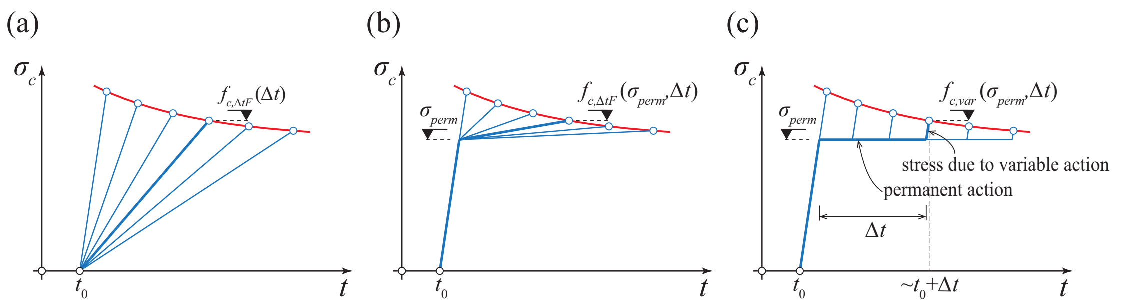

Fig. 4-7 Investigated loading patterns: (a) stress rate without initial loading; (b) stress rate with initial rapid loading; and (c) additional loading after a period of sustained loading

4.4.2 The Palmgren-Miner's rule for linear damage accumulation

The concept of linear damage theory was initially developed by Palmgren (Palmgren 1924) in 1924 and extended later by Miner (Miner 1945) in 1945, being widely used thereafter to describe the fatigue life of engineering materials (Oh 1991). Its main assumption considers that, when a material is subjected to a given stress amplitude, the damage accumulation is in linear correlation with the cycle ratio, i.e. $D = (N/N_F)$, where N refers to the number of cycles elapsed with a given stress amplitude and N_F is the number of cycles that leads to failure for that stress amplitude. For variable amplitude fatigue loads, the total accumulation of damage D would thus read:

$$D = \sum_{i=1}^{n} \frac{N(\sigma_i)}{N_F(\sigma_i)} \tag{4.14}$$

where i is the index of the i-th amplitude (stress level σ_i), and the occurrence of failure would be given when $D = 1$.

It is worth noting that Palmgren-Miner's formulation of damage does not account for the order in which the different stress levels are applied, which has been reported as a drawback of this rule (Oh 1991). Also, there is actually no thoroughly mechanical justification grounding it. Another reported drawback is the linearity assumption of damage accumulation for all stress levels (Miller et al. 1986). Hence, some authors have contested the applicability of the Palmgren-Miner's rule for concrete, especially in the case of variable amplitude loadings and complex loading patterns (Hilsdorf and Kesler 1966; Holmen 1982; Oh 1991; Zhang et al. 1996; Zanuy et al. 2010). Despite these drawbacks, the Palmgren-Miner's rule has still been widely used due to its simplicity and reasonable fitting to test results, and is for instance acknowledged in *fib* Model Code 2010 (fib 2013b) for detailed fatigue verifications.

With respect to the failure of concrete under variable sustained stress (low stress and strain rates), the Palmgren-Miner's rule can be applied by analogy to the case of fatigue life estimation under variable stress amplitudes, reading:

$$D = \sum_{i=1}^{n} \frac{\Delta t(\sigma_i)}{\Delta t_F(\sigma_i)} \; (= 1 \text{ at failure}) \tag{4.15}$$

where $\Delta t(\sigma_i)$ is the duration of the i-th level of sustained stress σ_i and $\Delta t_F(\sigma_i)$ is the associated failure time for the same constant level of sustained stress (normally only occurring for stress levels above $\sigma_c/f_c = 0.75$). The sum can also be written in an integral form in the following manner:

$$\sum_{i=1}^{n} \frac{\Delta t(\sigma_i)}{\Delta t_F(\sigma_i)} = \int_0^{\Delta t} \frac{dt(\sigma)}{\Delta t_F(\sigma)} \tag{4.16}$$

In the following sections, the Palmgren-Miner's rule will be used in combination with Eq. (4.13) proposed in section 4.3.4 for calculation of the time to failure (Δt_F) for a given level of sustained load.

4.4.3 Application to a constant stress rate with and without initial stress level

To apply the Palmgren-Miner's rule presented in section 4.4.2, the accumulation of damage can be integrated by means of a discretization of the loading history in time steps. *Fig. 4-8* shows for instance the case of a constant stress rate without and with an initial stress level (σ_{perm}, refer to *Fig. 4-8*a and *Fig. 4-8*b respectively). According to the Palmgren-Miner's rule, the first load steps generate low or no damage in the material. The stress level at failure for a given duration of the loading pattern is consequently higher than the one corresponding to a sustained loading pattern (refer to the solid and dashed lines in *Fig. 4-8*).When an initial stress level is applied (*Fig. 4-8*b), the first load steps generate more material damage and the failure stress is consequently closer to that of a constant sustained loading.

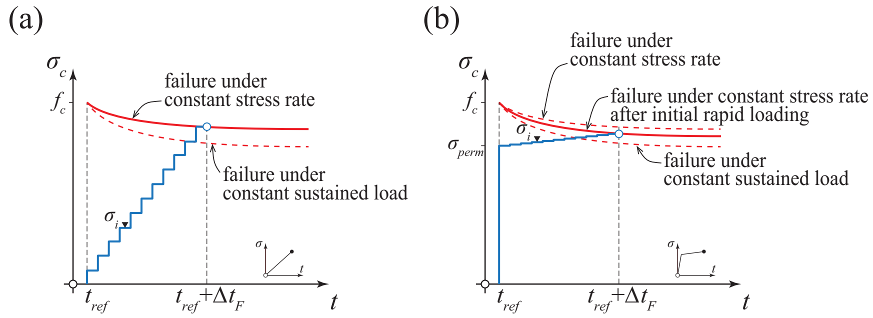

Fig. 4-8 Example of a numerical discretisation scheme for the Palmgren-Miner's rule for loading case of a constant stress rate: (a) no initial preload; (b) with initial preload

The results for these loading cases are investigated in *Fig. 4-9*a and *Fig. 4-9*b (case without rapid preloading and with a rapid preloading $\sigma_{perm} = 0.5 \cdot f_c$), comparing the numerical predictions of the mechanical model from Tasevski et al. (Tasevski et al. 2018) (solid lines) and the integration of Eq. (4.13) by means of the Palmgren-Miner's rule (dashed lines). The results show fine agreement for the different regimes: when the strength decays (damage governing, higher strain rates) and when the strength increases (continued cement hydration governing, lower strain rates). Values above one are also possible for significant strength increase due to continued cement hydration (typically associated to rather low strain rates). A comparison to the test results on variable stress rates from Tasevski et al. (Tasevski et al. 2018) (with and without rapid preloading) is also plotted in *Fig. 4-9*c and *Fig. 4-9*d, showing sound agreement for the different cases investigated. It can be noted that the results of the model are in good agreement with the test results even for very low times of application of the load, despite the fact that the creep model used (Tasevski et al. 2018) (*fib*'s MC2010) has not been calibrated to precisely describe these cases.

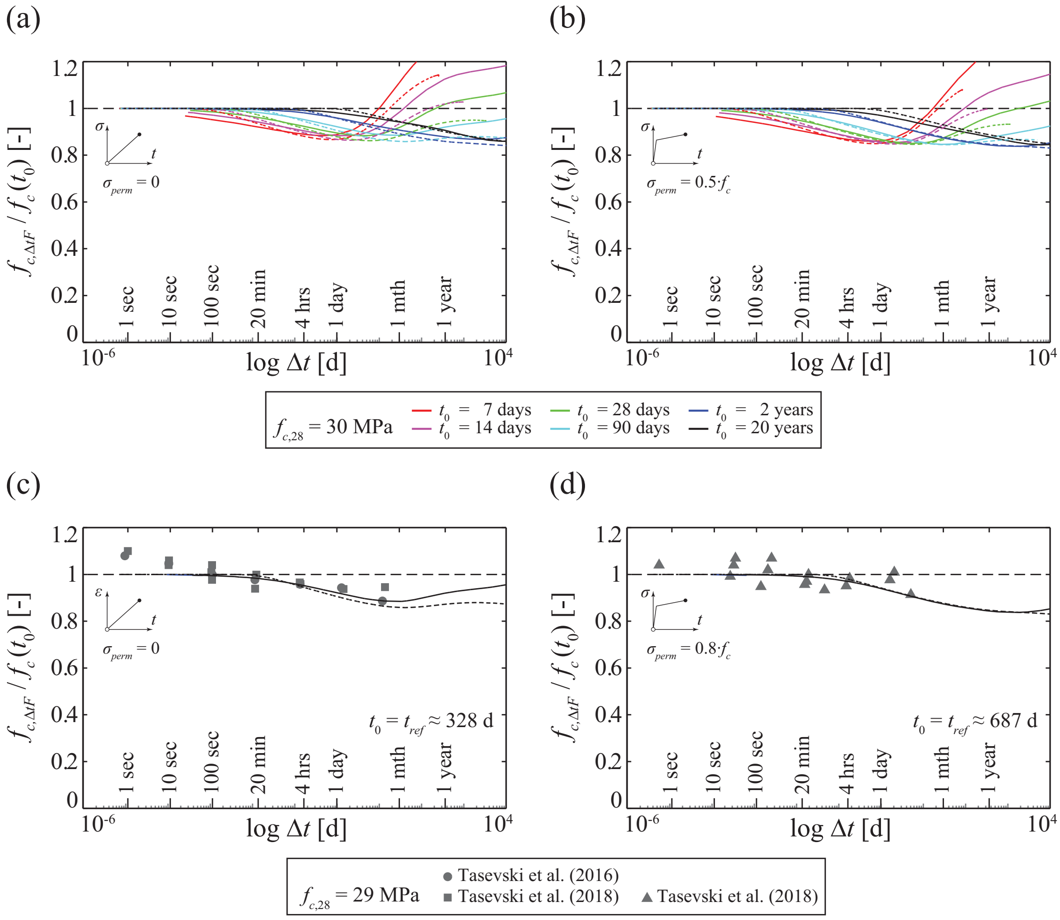

Fig. 4-9 Comparison of the theoretical model (Tasevski et al. 2018) (solid lines) and the analytical approach based on the Palmgren-Miner's rule (dashed lines) for constant stress rate with (a) no preload and (b) a preload of $\sigma_{perm} = 0.5 \cdot f_c$; comparison to test results from Tasevski et al. (Tasevski et al. 2018) for: (c) no preload and (d) a preload of $\sigma_{perm} = 0.8 \cdot f_c$. ($s = 0.25$)

4.4.4 Application of a rapid additional loading after a period of sustained load

Another loading pattern of practical relevance corresponds to the case when additional rapid loading is applied after a period of permanent sustained loading (*Fig. 4-7*c). When the levels of permanent load are high (*Fig. 4-10*a-c), material damage ($\varepsilon_{cc,in}$ in *Fig. 4-10*b) develops and the strength reduces accordingly (Fernández Ruiz et al. 2007) (see *Fig. 4-10*b), with reducing strength for increasing levels of damage. At failure, the total applied stress (σ_{tot}, comprising the effect of permanent and variable actions) equals the compressive strength accounting for the detrimental effect of sustained loading and the beneficial effect of continued cement hydration.

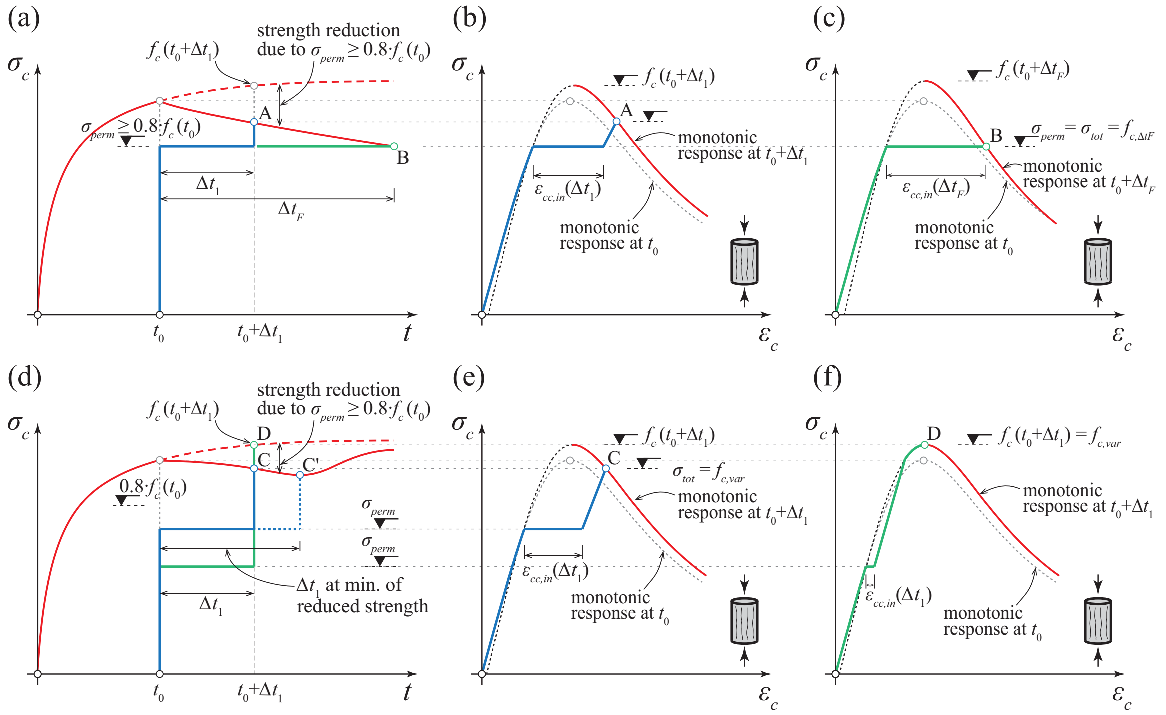

Fig. 4-10 Response of concrete for the application of a variable action after a period of sustained load: (a) loading patterns for σ_{perm} levels above the sustained load strength; (b-c) corresponding stress-strain responses; (d) loading patterns for σ_{perm} levels below the sustained load strength; (e-f) corresponding stress-strain responses

For practical purposes, two situations may be relevant with respect to this loading pattern:

- Application of the variable action after a permanent stress level (σ_{perm} in *Fig. 4-10*a) above the threshold of failure under sustained load. In this case (point A in *Fig. 4-10*a), large material damage occurs associated to reductions of the material strength (resulting in $\sigma_{tot} < f_c(t_0+\Delta t_1)$, see *Fig. 4-10*b). In any case, if the permanent action were applied for a sufficiently long period of time, a delayed failure of the material would occur even without the application of the variable action (point B in *Fig. 4-10*a and c).

- Application of the variable action after a permanent stress level (σ_{perm} in *Fig. 4-10*d) below the threshold of failure under sustained load. In this case, if the stress level is sufficiently low, the material will suffer no (or very limited) damage and no strength reduction will be apparent ($\sigma_{tot} = f_c(t_0+\Delta t_1)$), point D in *Fig. 4-10*f). However, in some cases (for permanent stress levels close to the sustained load strength, *Fig. 4-10*e), a certain material damage can occur and this can potentially lead to a reduction of the material strength when the variable action is applied (point C in *Fig. 4-10*d,e). In this case, as the continued cement hydration eventually compensates for the material damage due to the sustained loading, a minimum value to the strength can be found (represented as point C' in *Fig. 4-10*d).

For the former situation (*Fig. 4-10*a-c), the envelopes of the reduction of strength (points A) for a constant level of permanent stress (σ_{perm}) are plotted in *Fig. 4-11*a. The case corresponding to the permanent stress level equal to the sustained load strength is represented by the green curve, while cases with higher levels of permanent load are plotted as grey curves. The dark grey area in the top right corner of *Fig. 4-11*b denotes thus the area where a failure under permanent load is possible even if no variable actions were applied.

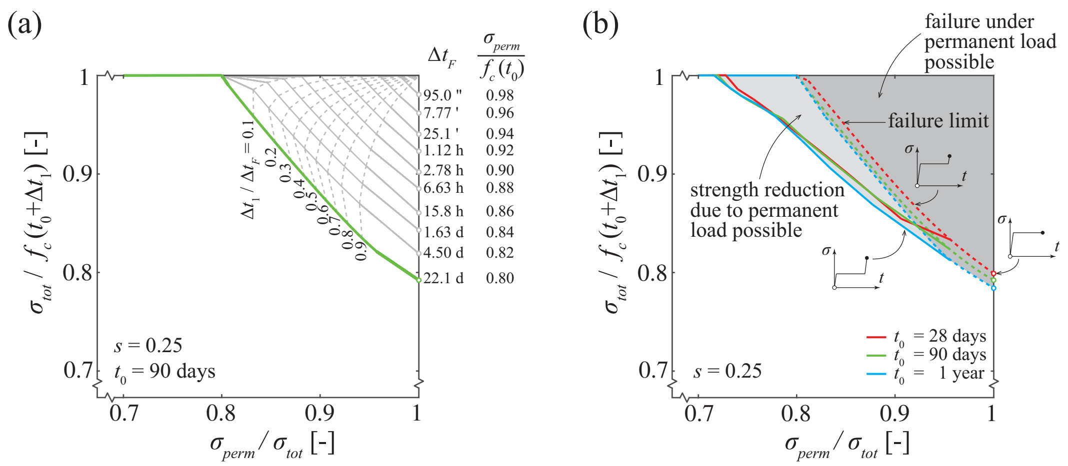

Fig. 4-11 Response of concrete for the application of variable action after a period of sustained load: (a) envelopes of the reduction of strength for σ_{perm} level above the sustained load strength; (b) areas of danger of strength reduction with and without failure danger.

It is interesting to note that similar shapes of the curves (almost linear) and strength reductions are found for other times of application of the permanent load (t_0, refer to the red and blue dashed curves in *Fig. 4-11*b).

For the case of permanent stress levels below the sustained load strength of concrete (no delayed failure under only permanent load), the solid curves in *Fig. 4-11*b represent the envelopes of minimum material resistance after application of the variable action (points C' in *Fig. 4-10*d). The shape of these curves can again be reasonably approximated by a linear segment, defining the region where a reduction on the concrete strength can potentially happen due to the damage developed during the application of the permanent stress level (refer to the area shaded in light grey in *Fig. 4-11*b). It is interesting to note that for ratios $\sigma_{perm} / \sigma_{tot} < 0.75$ no strength reduction is to be accounted for. Above this threshold, safe design can be performed by considering a linear reduction (up to 20%) of the uniaxial compressive strength. This can be considered in the format of a design expression as follows:

$$\frac{\sigma_{tot}}{f_c(t_0+\Delta t_1)} = 1.0 \quad \text{if } \frac{\sigma_{perm}}{\sigma_{tot}} \leq 0.75$$

$$\frac{\sigma_{tot}}{f_c(t_0+\Delta t_1)} = 1.6 - 0.8\frac{\sigma_{perm}}{\sigma_{tot}} \quad \text{if } \frac{\sigma_{perm}}{\sigma_{tot}} > 0.75 \tag{4.17}$$

It can be noted that for total levels of stress $\sigma_{tot} \leq 0.75 \cdot f_c(t_0+\Delta t_1)$, there is no strength reduction.

4.5 Practical design considerations

4.5.1 Implications for codes of practice

In codes of practice, the design compressive strength of concrete is used in a number of situations such as (i) the resistance of axially-loaded members (e.g. columns), (ii) the compression zone due to bending, (iii) the compression field due to shear in webs of girders and beams, (iv) the strut resistance when designing using strut-and-tie models or stress fields and (v) partially loaded areas (e.g. introduction of bearing or prestressing forces). For all these cases, design equations used in codes of practice have been validated and calibrated on the basis of laboratory tests. Since these tests are usually conducted in a period varying between approximately 20 minutes and some hours, it can be stated that the structural design formulas for verification at ultimate limit state already partially account in an implicit manner for the detrimental influence of low stress or strain rates. *Fig. 4-12* shows for instance that for typical test durations and typical specimen ages, it can be assumed a compressive concrete strength decrease between 4% - 8% compared to a material test duration of approximately 2 minutes (according to ISO 1920-4:2005 (ISO 2005)). For this reason, the strength ratio at failure previously discussed ($\sigma_{tot} / f_c(t_0+\Delta t_1)$) may be corrected by increasing it by approximately 6% when formulas for design of structural elements are used. Thus, in the previous Eq. (4.17), this effect can be accounted for by shifting the curve in the following manner:

$$\frac{\sigma_{tot}}{f_c(t_0+\Delta t_1)} = 1.0 \quad \text{if } \frac{\sigma_{perm}}{\sigma_{tot}} \leq 0.85$$

$$\frac{\sigma_{tot}}{f_c(t_0+\Delta t_1)} = 1.85 - \frac{\sigma_{perm}}{\sigma_{tot}} \quad \text{if } \frac{\sigma_{perm}}{\sigma_{tot}} > 0.85 \tag{4.18}$$

where design values for the stress and material strength shall be considered (*Fig. 4-1*b). As a consequence, for the case $\sigma_{perm} / \sigma_{tot} = 1$, the limit stress value shifts from 0.80 (Eq. (4.17)) to 0.85 (Eq. (4.18)). It can also be noted that, for practical purposes, the stresses indicated in Eq. (4.18) are normally replaced by a generalized stress (internal forces).

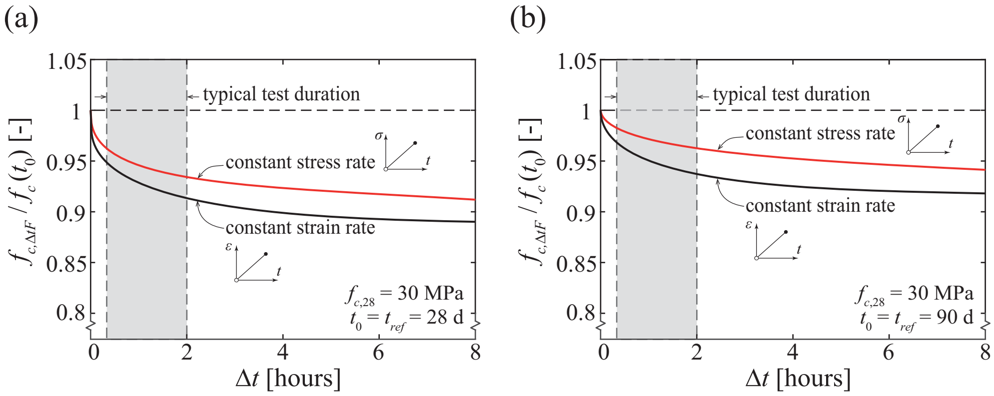

Fig. 4-12 Influence of test duration on the compressive strength of concrete for constant stress and strain rates for: (a) t_0 = 28 days; (b) t_0 = 90 days (calculated according to the approach of Tasevski et al. (Tasevski et al. 2018))

4.5.2 Design versus assessment

As already introduced, for design of new structures, codes of practice usually assume that the detrimental effect of sustained loading is compensated by the strength increase due to continued cement hydration. This fact is investigated in *Fig. 4-13* for the most critical loading case ($\sigma_{perm} / \sigma_{tot} = 1$) and with reference to different types of cement (defined by the parameter s characterizing the rheology of cement hydration and thus the increase of concrete strength with time (fib 2013b)). The results are calculated on the basis of the refined rheological approach by Tasevski et al. (Tasevski et al. 2018) and assuming Eq. (4.10) for calculation of the continued cement hydration (refer to black curves in the diagrams for the material compressive strength for rapid loading). In the figures, several cases are presented corresponding to the application of a sustained load at different ages. It can be observed that, for a given t_0, the strength decreases as the time of application of the sustained load increases, reaching an envelope curve (red curves in *Fig. 4-13*) at approximately 80% of the rapid loading strength.

It can be noted that the assumption that the detrimental effect of the sustained load on the compressive strength is compensated by the continued cement hydration is valid only when the time of load application t_0 is sufficiently large (refer to the construction sequence in *Fig. 4-1*b). With this respect, the areas shaded in light grey in *Fig. 4-13* refer to the period when the sustained loading effect is not fully compensated by the continued cement hydration. In addition, the time when the concrete strength is not fully compensated depends significantly on the value of parameter s. For instance, considering a $t_{ref} = 28$ days (see *Fig. 4-13*), the full compensation of the strength occurs after 4 months for $s = 0.35$ (low early strength class concrete) and two years for $s = 0.25$ (ordinary early strength class concrete). For the case of $s = 0.15$ (very high early strength class concrete), the continued cement hydration does not even appear to fully compensate for the effects of sustained loading.

With this respect, and as previously discussed, it shall be noted that for design of structural members (columns, bending, shear in webs…) the detrimental effect of sustained loading is already partially accounted for in the design equations. This implies that, for design of structural members, the period where the continued cement hydration does not fully compensate for the sustained loading effect can in fact be assumed shorter than for the material response. This can be seen in *Fig. 4-13*, where the corrected ratio between the failure under sustained load strength (1/1.06 discussed in section 4.5.1) is also plotted.

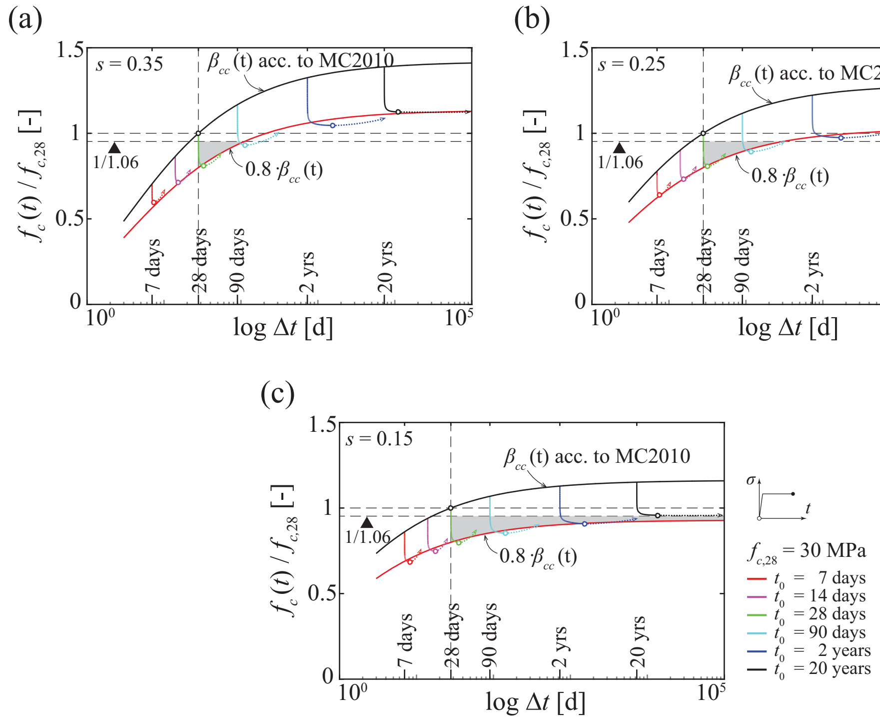

Fig. 4-13 Influence of the age of loading on the delayed failure under sustained load ($f_{c,28}$ = 30 MPa): (a) s = 0.35 (low early strength class concrete); (b) s = 0.25 (ordinary early strength class concrete); and (c) s = 0.15 (high early strength class concrete)

For assessment of existing structures, the approach to be followed is very different to that for design. The concrete strength is in this case usually updated by means for instance of core samples extracted from the actual structure at a time t_{ref} much larger than 28 days (normally some years or decades after construction). Consequently, the concrete strength increase due to continued cement hydration can be reasonably neglected and, if the sustained loading is governing ($\sigma_{perm} \approx \sigma_{tot}$), then a conservative value $\sigma_{tot} / f_c(t_0+\Delta t_1)$ = 0.85 (≈0.8·1.06) should be assumed. This assumption is obviously safe and when variable (rapid) actions may have significant relevance, a more refined estimate of the strength can be obtained by means of Eq. (4.17) and Eq. (4.18) (material and structural levels respectively).

For more complex loading patterns or when more accurate estimates of the strength reduction shall be obtained (both for design and for assessment), the general approaches described in this paper (using the Palmgren-Miner rule or the general rheological approach by Tasevski et al.(Tasevski et al. 2018)) can for instance be used.

4.5.3 Influence of time of application of variable loads on the strength of concrete

The case of application of a variable action after a period of sustained load was investigated in section 4.4.4 by considering the variable action as instantaneous. However, this might not be the case in many design situations, where the variable actions might be applied during some minutes, hours or even days (refer to point E in *Fig. 4-14*a).

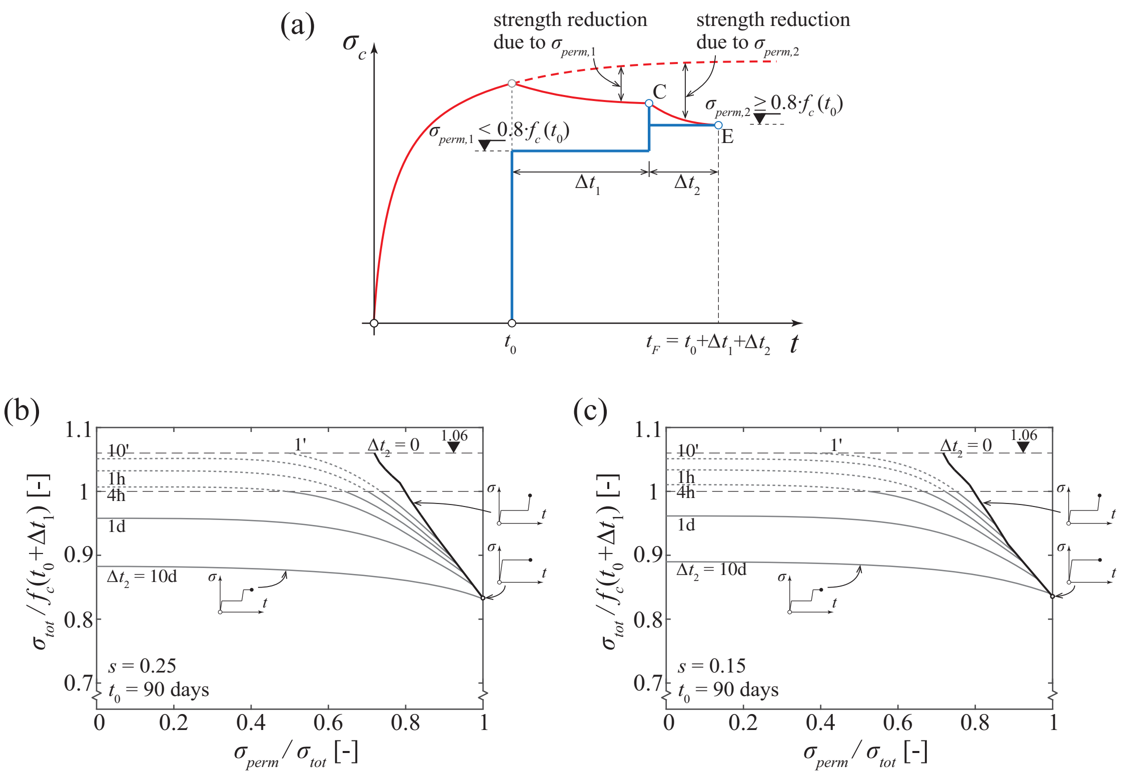

Fig. 4-14 Influence of the duration of the application of the variable load: (a) load pattern; (b-c) strength reductions (for structural design) as a function of the duration of the variable action

In a general manner, the influence of the duration of the application of the variable load on the concrete strength can be addressed by using the general procedures presented in this paper. *Fig. 4-14*b,c show for instance the results calculated by using the refined model of Tasevski et al. (Tasevski et al. 2018) with reference to two values of parameter s (0.25 and 0.15 respectively, values for $s = 0.35$ being almost identical to those of $s = 0.25$). In those figures, the factor increasing the strength by 6% has already been considered, in order to be consistent with the application of these results to design formulas (referring normally to generalized stresses or internal forces). The results show that a reduction on the strength can already be noted for relatively low times of application of the variable load and that it can clearly be appreciated for durations of the variable load of some days. The ratio $\sigma_{perm} / \sigma_{tot}$ also plays a significant role, by reducing notably the strength for ratios $\sigma_{perm} / \sigma_{tot}$ above 0.6.

4.6 Conclusions

This paper investigates on the reduction of the strength of concrete when it is subjected to high levels of sustained stress and to different loading patterns. A simplified analytical approach is proposed based on the general mechanical model by Tasevski et al. (Tasevski et al. 2018). The main conclusions of this paper are:

1. The phenomenon of concrete creep shall be accounted for both for Serviceability Limit States (SLS) and Ultimate Limit States (ULS). Even if at SLS the ratio between the applied stresses and the material strength is moderate, at ULS this ratio might be high accounting

for the characteristic values of actions and material strengths and for the safety coefficients. In this latter case, material damage may develop (characterized by microcrack progression and coalescence) and lead to failures under sustained load.

2. The development of inelastic strains associated to material damage due to the application of high levels of stress is potentially detrimental for the strength of a member, unless they can be compensated by the increase of strength due to the continued cement hydration. This material damage is associated to the development of nonlinear creep strains which however have a potentially favourable effect for the structural strength, allowing for stress redistributions between more and less stressed regions. This latter phenomenon is particularly beneficial and notorious for compression regions in bending (with a strain gradient) or for reinforced concrete columns, where the longitudinal reinforcement can be additionally activated.

3. Simple expressions can be derived to characterize the failure load of concrete under sustained stress. Combining these expressions with the Palmgren-Miner's rule provides a practical manner to account for damage accumulation and to estimate the failure load and time. When compared to test results, both a detailed material-damage approach and the simplified Palmgren-Miner's rule yield fine and consistent results for stress and strain rate loading patterns.

4. The analyses presented in this paper confirm that the beneficial effect of continued cement hydration usually compensates for the detrimental effect of sustained loading. This holds true provided that the concrete strength is determined at 28 days and the ULS occurs after a sufficiently long period of time. In case the concrete strength is determined later than 28 days (particularly relevant for assessment of existing structures), the reduction of concrete strength accounting for long-term and high sustained loading is justified.

5. When concrete is loaded some weeks after casting, the results of this paper confirm that failures under sustained loading typically occur after some hours or days. However, when a concrete is loaded at very high age (some years or decades), failures may potentially occur after a longer period of time.

6. For a given level of total stress, a combination of permanent actions and a rapid variable action is less detrimental for the concrete compressive strength than a full permanent action. This phenomenon can be described by simple, code-like expressions resulting in a linear interpolation between the response of a member failing under constant sustained load and the case where permanent load effects can be neglected. Also, for cases where the variable action is maintained over a period of time, a similar approach can be followed (associated to larger strength reductions than rapid applications of the variable loads).

4.7 Notation

Term	**Signification**
Variables	
f_c	uniaxial compressive strength of concrete [MPa]
$f_{c,tref}$	reference compressive strength obtained at a strain rate of 0.02 ‰·s^{-1} and at an age t_{ref} [MPa]
$f_{c,28}$	reference compressive strength at an age of t_{ref} = 28 days [MPa]
$f_{c,\Delta tF}$	compressive stress at failure under sustained load or stress/strain rate [MPa]
$f_{c,var}$	compressive stress at failure under variable action after a permanent load σ_{perm} [MPa]
k_1, k_2	constants
k_t	strength reduction factor of concrete accounting for sustained load action
t	time
t_0	concrete age at loading [days]
t_{ref}	concrete age at reference strength testing [days]
Δt	time under sustained stress for a constant stress level test or time after beginning of a strain or stress rate test (in the case of rapid initial loading, time after the initial loading ramp)
Δt_F	time under sustained stress required to attain failure
α	parameter governing the shape of tertiary creep strain development curve
β_{cc}	parameter to describe the development of concrete strength with time acc. to MC 2010
γ	parameter characterizing tertiary creep
$\dot{\varepsilon}$	strain rate
ε_c	concrete strain
ε_{c0}	instantaneous pre-peak strain
ε_{cc}	creep strain
ε_{cs}	shrinkage strain
φ	creep coefficient
λ	constant in [days]
η, η_τ	affinity coefficient
σ_c	concrete stress
σ_{perm}	stress due to permanent actions
σ_{tot}	total applied stress
Indexes	
av	available
c	concrete
cc	concrete creep, except in the case of β_{cc}
cc,1	primary creep
cc,2	secondary creep

cc,3	tertiary creep
cs	shrinkage
F	at failure
in	inelastic
lin	linear
nl	nonlinear
perm	permanent load
ref	reference
tot	total
τ	time-related
var	variable action

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Chapter 5 Influence of load duration on shear strength of reinforced concrete members

This chapter presents a detailed investigation of the influence of the loading rate on the strength of shear-critical reinforced concrete members without transverse reinforcement. The results of an experimental campaign on slender and squat beams are presented, with loading rates ranging from several seconds to several months until failure. The development of the failure mechanism is discussed and a detailed investigation is performed on the observed development of crack patterns with time. Furthermore, the contribution of the different shear transfer actions is evaluated. The observations lead to a validation of an existing model for shear strength prediction (CSCT) and help to draw several practical design recommendations.

The chapter is a preprint version of a publication submitted to a peer-reviewed academic journal. The authors are Darko Tasevski (PhD Candidate), Miguel Fernández Ruiz (Senior lecturer and thesis director) and Aurelio Muttoni (Professor and thesis director), and the submission reference can be found here:

> **Tasevski, D., Fernández Ruiz, M. and Muttoni, A., (2019),** "Influence of load duration on shear strength of reinforced concrete members.", *submitted for publication*.

The main contributions of Darko Tasevski are the following:

- Preparing and performing the full experimental campaign
- Detailed analysis and interpretation of the experimental findings
- Development of the numerical tools for post-processing of experimental observations
- Analytical study of failure mechanisms and shear transfer actions
- Production of the artwork in the article
- Preparation of the manuscript of the article

The second and third author (thesis directors) contributed to the understanding of the studied phenomena, interpreting of the experimental and analytical findings and writing of the final manuscript.

5.1 Abstract

The significance of sustained loading on the compressive and tensile strength of concrete has been experimentally verified since the 1950s and is currently acknowledged in most design codes implicitly or explicitly. However, its influence on other potentially sensitive phenomena, as for instance the case of members subjected to shear, has not yet been clearly established. Currently, there is still scanty experimental data available on long-term shear tests and they present inconclusive results to assess whether high levels of sustained load have a detrimental effect on the shear strength. In order to advance on the knowledge of the phenomenon, this manuscript presents the results of an experimental programme on sixteen reinforced concrete full-scale beams without shear reinforcement tested under varying loading rates (associated to different times to failure, from some seconds to some months). The programme consists of two series, one dedicated to slender beams and another to squat members (with different potential significance of the arching action in their shear response). The results show no significant reduction on the shear strength for low loading rates (long times of application of load) in any of the two tests series investigated. Yet, some enhancement is observed in the shear strength for high loading rates (failures in some seconds) compared to typical test durations. The experimental results are analysed and discussed with reference to refined measurements performed during the tests in terms of crack shape and development. These observations, together with the evaluation of the contributions of the potential shear transfer actions in the specimens, lead to a series of practical design considerations.

5.2 Introduction

Civil engineering structures are subjected to loads of diverse durations ranging from rapid actions (as loads on a bridge deck slab) to loads sustained over longer periods of time (as self-weight actions or soil pressure). Since the early developments of concrete, significant research efforts have been devoted to the topic of sustained loading and loading rates, mostly in terms of the compressive and tensile behaviour of concrete. Yet there exists still very little knowledge on the influence of these phenomena on the shear strength of concrete members. This situation might be explained by the fact that sustained loading has traditionally been associated to the serviceability limit state (SLS) of concrete structures, and thus mostly related to the linear response of the material (Ghali et al. 2002; Gilbert and Ranzi 2011).

However, as early acknowledged by Shank (Shank 1949) and later by Rüsch (Rüsch 1960), high levels of sustained loading influence also the compressive strength of concrete with the development of nonlinear creep strains. These nonlinear creep strains are associated to development of material damage (microcrack propagation and growth) and reduce the strength with respect to short-term loading. Since these pioneer works, the time-dependent behaviour of concrete in the nonlinear domain of material response has been widely investigated, both in terms of compressive as well as tensile behaviour (extensive literature review can be found elsewhere (Müller et al. 2010; Fischer et al. 2014; Tasevski et al. 2018, 2019)).

With respect to the shear strength of reinforced concrete members, this topic has followed an intensive research since the early concrete developments (fib 2010). In the last decades, traditional empirical approaches for design of members without shear reinforcement (Zsutty 1968) are being increasingly replaced by models based on physical principles and accounting for the role of the various potential shear transfer actions (see state-of-the-art on this topic reported in (fib 2010)). Recently, the potential of advanced measurement techniques based on Digital Image Correlation (DIC) are also becoming instrumental to understand the role of each shear transfer action (Cavagnis et al. 2015; Huber et al. 2016) and how they contribute to the shear resistance depending on the crack shape and kinematics (Cavagnis et al. 2018b).

Despite the significant efforts performed in the past on both domains (long-term uniaxial response of concrete and short-term shear strength of concrete members), few works have been addressed to the understanding of the shear strength under sustained loading. In terms of experimental evidence, the first comprehensive test series dedicated to this issue was performed by Sarkhosh et al. (Sarkhosh 2013, 2014). This experimental programme was performed on shear critical slender beams (a/d = 3.0) under sustained loading, focusing on the investigation of the time-dependent crack growth. The study comprised 42 simply supported beams without shear reinforcement, with 18 specimens subjected to high levels of sustained loads (87% - 95% of the average short-term shear capacity). Two of these beams failed under sustained loading (one after 2.5 hours and another after 44 hours). The beams that did not fail under sustained loading were eventually reloaded to failure, and showed in general a higher resistance than the reference beams. The results of Sarkhosh are presented in *Fig. 5-1*a,b and compared to the prediction of the Critical Shear Crack Theory (CSCT) formulated according to (Muttoni and Fernández Ruiz 2008) for reference loading rates (failures occurring for loads applied during a period of 5 minutes to 2 hours). For this analysis, the equivalent aggregate size d_g is calculated according to (Cavagnis et al. 2018a) and the material strength is considered at the time of failure (details on the fundamentals of the theory can be found elsewhere (Fernández Ruiz et al. 2015)). The results show no significant decay on the strength and most specimens required to be reloaded to reach failure (failing at higher load levels than the reference specimens).

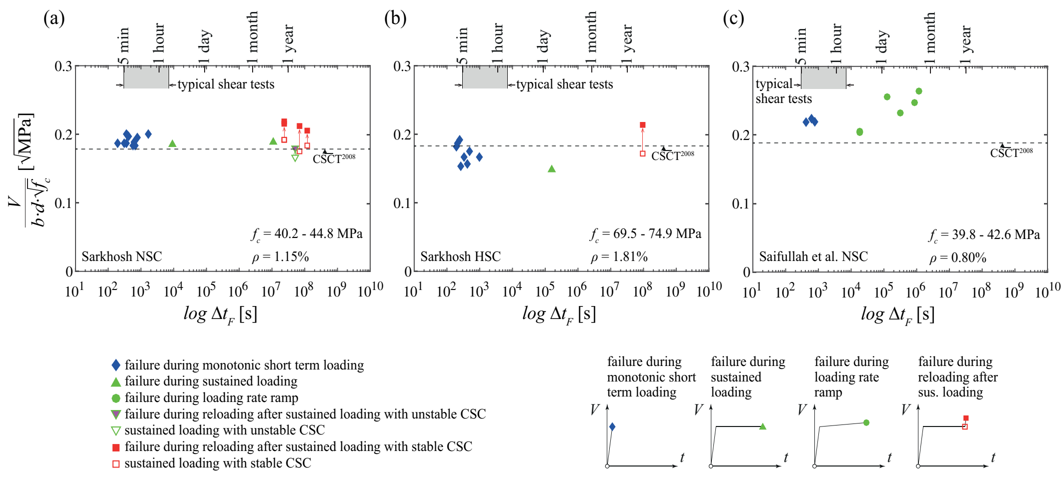

Fig. 5-1 Tests from literature on the influence of high levels of loading on the shear strength and comparison with the CSCT2008 (Muttoni and Fernández Ruiz 2008): (a-b) Sarkhosh (Sarkhosh 2014); and (c) Saifullah et al. (Saifullah et al. 2017)

Another recent study on the effects of sustained loading on the shear strength of slender concrete beams (a/d = 3.0) has been performed by Saifullah et al. (Saifullah et al. 2017). In this series, the influence of low loading rates was investigated, comprising loading rates 100 to 1000 times lower than the reference loading rate (typically requiring about an hour to reach the shear failure). Three beams were tested under reference loading rate and 6 beams under low loading rates. A general increase of midspan deflection, crack opening and compressive strain at the top fibre with increasing time to failure was observed, as well as an overall increase in the ultimate shear capacity (yet the results were somewhat scattered, see *Fig. 5-1*c).

With respect to squat members, where arching action can be governing and thus a different shear response can be expected (Sagaseta and Vollum 2010; Cavagnis et al. 2018b), there is still a lack of experimental studies. It can however be cited the works of Maekawa et al. (Maekawa et al. 2016) to monitor the long-term deformation of underground box culverts, with the potential detection of a delayed shear failure under sustained soil pressure (confirmed by inspection of boreholes showing the existence of a well propagated diagonal shear crack). In a further numerical study on squat beams (a/d = 1.0 – 2.4), Bugalia and Maekawa (Bugalia and Maekawa 2017) showed that arching action and direct strut action can potentially be influenced by sustained loading.

The scanty studies in the field of shear under sustained loading is also reflected in codes of practice, where there is no clear consensus on the consideration of this effect. While ACI 318-14 (ACI 2014), Eurocode 2 (CEN 2004) and Model Code 2010 (fib 2013b) acknowledge the effect of sustained loading on the compressive strength of concrete (which is considered in an implicit or explicit manner or explicitly neglected since it can be compensated by the strength increase due to continuous cement hydration (fib 2013b)), no clear reference is provided in the shear section for slender members. However, for squat members verified with strut-and-tie models, this effect is normally implicitly accounted for in the strength of the struts. Other codes, as for instance the current Swiss code for concrete structures SIA 262 (SIA 2013), accounts for long-term effects both for slender and squat members when the concrete strength is evaluated at ages older than 28 days or for early loading times. According to SIA 262, a reduction on shear strength for high levels of sustained loading shall be introduced, in a similar manner than for the compressive resistance of the concrete.

Within this context, the aim of the present study is to provide new experimental data on concrete members without shear reinforcement subjected to high levels of sustained loading. The experimental programme consists of two series, one dedicated to slender members (a/d = 3.5) and the other to squat ones (a/d = 1.0) in order to understand the potential differences in the governing shear transfer actions. The phenomenon is investigated by means of low loading rates and by performing refined experimental measurements (based on DIC) to track the cracking patterns and associated kinematics. On this basis, the contribution of the different shear transfer actions is investigated and a discussion is performed on the need for a reduction on the shear strength accounting for the duration of the loading. For comparison purposes, some specimens are also tested under relatively high loading rates and their results are also discussed.

5.3 Research significance

Although a significant fraction of shear-critical structures is potentially subjected to high levels of sustained loading at ultimate limit state and the detrimental effect of this action on the compressive and tensile strength of concrete is well documented, very few studies have been devoted to the topic of the shear resistance under long-term actions. In addition, there is no consensus on design codes on how this phenomenon shall be addressed. In order to improve current state-of-the-art, this paper presents the results of a testing programme consistently investigating this phenomenon both for slender and squat members. Tests with durations ranging from some seconds to some months are presented and their results, investigated by means of refined experimental measurements, are used to draw a number of practical design considerations.

5.4 Experimental programme

Sixteen beams have been tested in total, divided in two series, the first one comprising eight slender beams (shear span to effective depth ratio $a/d = 3.5$) and the second one comprising eight squat beams ($a/d = 1.0$). The failure process and the development of the corresponding cracking patterns have been observed in detail by means of Digital Image Correlation (DIC) measurements.

5.4.1 Materials and specimens

The specimens investigated were reinforced concrete beams with a rectangular cross section ($b \times h = 250 \times 600$ mm) and a flexural reinforcement ratio $\rho = 1.33\%$. The flexural reinforcement was composed of 3 bars diameter 28 mm placed at an effective flexural depth of 556 mm. The reinforcement was made of high-strength cold worked steel with an average yield strength $f_y = 713$ MPa (corresponding to the 0.2% proof stress (CEN 2004)). Neither shear nor compressive reinforcement were used.

Normal-strength concrete was used for all specimens. The average compressive strength at 28 days (measured in cylinder $Ø \times h = 160 \times 320$ mm) resulted $f_{c,28} = 29$ MPa. It was produced with a CEM-II 42.5R cement ($w/c = 0.56$) and river aggregates (maximal aggregate size $d_g = 32$ mm). The beams were cured during 18 days and stored under controlled-ambient laboratory conditions until the day of testing (temperature of 21±0.5 °C and relative humidity of 65±3 %).

The concrete strength at the time of testing for each series and specimen can be consulted in *Table 5-1* and *Table 5-2*. *Fig. 5-2* plots the development of concrete strength, shrinkage strains and linear creep coefficient with time (details for definition and measurements of shrinkage strains and creep coefficient on specimens from the same concrete batch can be consulted in Tasevski et al. (Tasevski et al. 2018)).

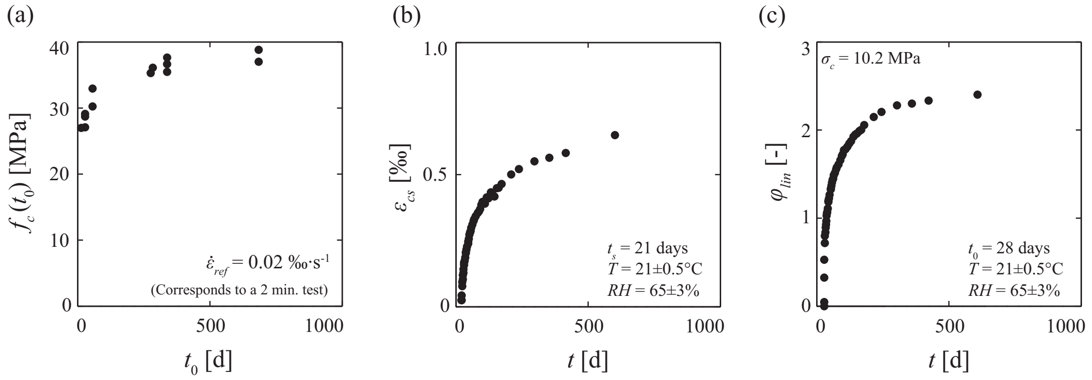

Fig. 5-2 Development of: (a) compressive strength; (b) shrinkage strain; and (c) linear creep coefficient of the concrete used in the experimental programme of this paper

5.4.2 Test setups

The test setups are shown in *Fig. 5-3*. The slender beams were tested as cantilevers where the test specimens were clamped to the testing frame by prestressing four vertical bars diameter 50 mm (see *Fig. 5-3*b) whereas the squat beams were tested as simply supported members. In both cases, the load was applied at the end support of the beams with a servo-hydraulic jack (capacity 2000 kN) pushing upwards. In the case of the squat members, the beams were retained in the middle by four bars diameter 50 mm.

The shear force was measured at the load introduction between the jack and the beam by means of a load cell (capacity of 500 kN, 0.1% noise level). Additional measurements were conducted with load cells at the vertical prestressing/retaining bars and between the beam and the testing frame in the case of the slender members. Both measurements were compared with consistent agreement (differences lower than 4%). In the following, the measurements of the shear force will be given between the jack and the beam at the end support. All tests were performed under controlled-ambient laboratory conditions (same conditions than before testing).

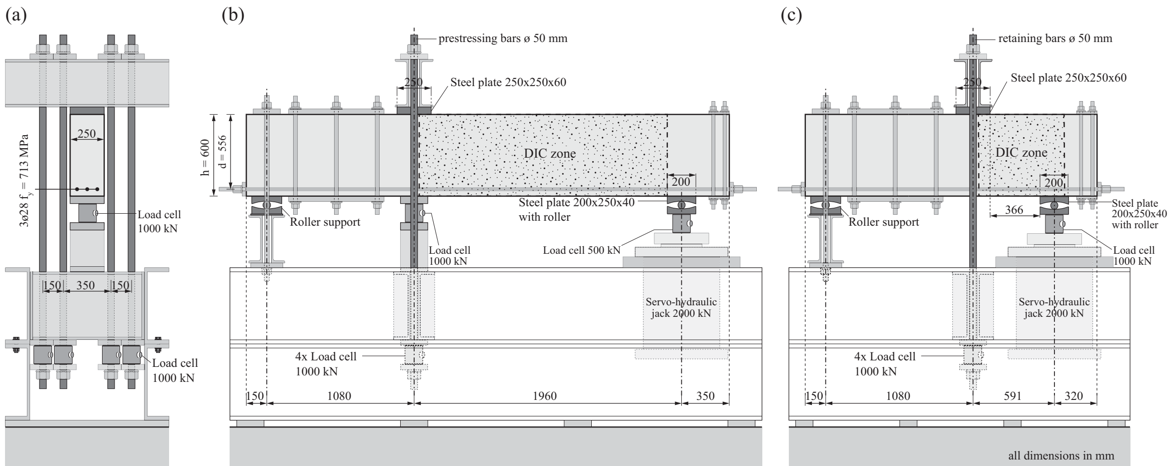

Fig. 5-3 Test setup: (a) section; (b) side view for slender beams; and (c) side view for squat beams

Photogrammetric (DIC) measurements setup

The development of the cracking pattern in the shear critical zone was tracked by means of 3D DIC (*Fig. 5-3*). The DIC hardware consisted of a pair of CMOS sensor cameras (model jAi SP-20000-USB, 20 Mpix), which, in combination with green lighting, had low sensitivity to day-night luminosity changes for the long-term tests.

The image acquisition was performed every 2 kN (ca. 1/100 of the failure load) for the slender beam series and 4 kN (ca. 1/250 of the failure load) for the squat beam series. Close to failure, the acquisition of images was increased to a frequency of 8 Hz.

A random speckle pattern with an approximately constant density was applied on the beam surface by means of spray painting. Depending on the camera distance, the size of the speckles was 1.5±0.5 mm for the slender beam series and 0.8±0.3 mm for the squat beam series (the physical pixel size was of approximately 0.36 mm and 0.25 mm respectively). The deformation analysis was performed with the Vic3D software (Correlated Solutions (CorrelatedSolutions 2017)), using a subset size of 23 × 23 pixels and 21 × 21 pixels for the slender and squat beam series respectively. The stable ambient control and the consistent calibration procedure helped to keep the error of displacement measurements below 1/75 of a pixel.

Loading patterns

Two loading patterns were used. The first one corresponds to a constant displacement rate at the load introduction at the end support up to failure, applied on specimens RT1.05 - RT5.01 of the slender beam series and specimens RT11.24 – RT15.17 of the squat beam series (for specimen RT15.17 the deflection rate was however changed to a lower one at V_{exp} = 250 kN).

The second type of loading pattern consisted of a preloading at a constant displacement rate up to a certain load level, followed by a constant load rate afterwards. For the slender beams, the change of the loading condition was applied at the moment when the flexural cracks started to develop in a quasi-horizontal manner above the neutral axis (120 kN for RT6.06 and RT8.18 and 100 kN for RT7.19). For the squat beams (RT16.15 – RT18.25), the change of the loading condition was applied at V_{exp} = 800 kN, which was the level at which the diagonal cracking developed. Details on the actual loading path in terms of applied shear force with time and corresponding deflection at the load introduction are provided in the following sections.

With respect to the high loading rates, failure occurred after some seconds of application of load (3.88 seconds as minimum loading time). Although such time was short, it allows neglecting any significant inertial effect in the results.

5.4.3 Results for slender beams (*a*/*d* = 3.5)

The time to failure and the measured shear strength of the slender beams are presented in *Table 5-1*. *Fig. 5-4* plots the normalized measured shear force versus the logarithm of time (*Fig. 5-4*a) and the deflection at the load introduction at the edge support (*Fig. 5-4*b). For low loading rates, a roughly constant normalized strength was observed. For higher loading rates, an increase on the shear strength was rather consistently observed. Overall, a reduction on the normalized shear strength of approximately 16% was measured between the highest (t_F = 3.88 s) and the lowest (t_F = 73.8 days) loading rates.

Table 5-1 Overview of the results of the slender beam series (round bracket value corresponds to first peak of specimen RT1.05, t_0 is the age of concrete at load application and t_F is the loading time at failure)

Beam	t_0	$f_c(t_0)$	t_F	$t_{F,100\text{kN}}$	$V_{exp} / (b \cdot d \cdot \sqrt{f_c})$	V_{exp}
	[days]	[MPa]	[-]	[-]	[√MPa]	[kN]
RT1.05	359	36.4	3.88 (2.41)	2.87 (1.40)	0.262 (0.232)	219 (194)
RT2.04	352	36.4	11.4 s	6.87 s	0.240	200
RT3.03	327	36.3	104 s	60.2 s	0.222	186
RT4.02	283	36.0	19.5 m	12.0 m	0.238	200
RT5.01	268	35.9	5.70 h	3.08 h	0.213	179
RT6.06	460	36.8	1.01 d	0.88 d	0.195	165
RT7.19	494	36.9	13.8 d	13.7 d	0.207	175
RT8.18	1004	37.7	73.8 d	73.6 d	0.201	171

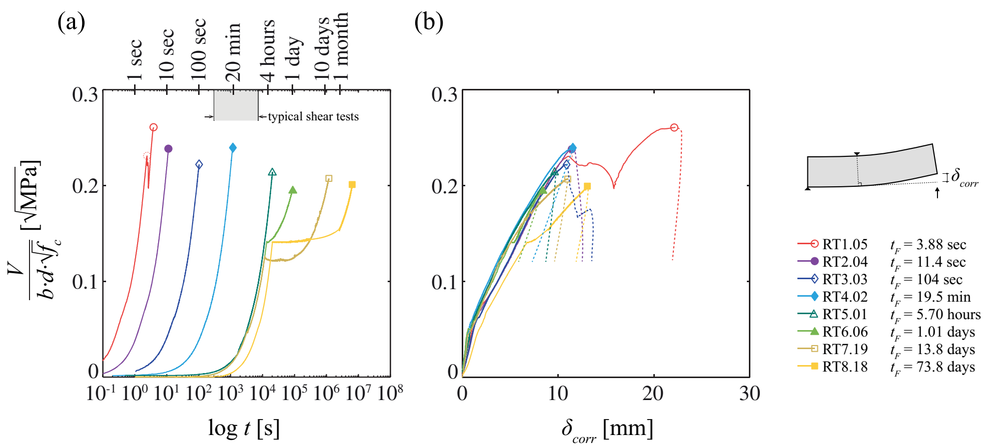

Fig. 5-4 Results of the slender beam series (a/d = 3.5): (a) normalized shear force vs. time in logarithmic scale; and (b) normalized shear force vs. deflection at the load introduction (corrected by removing the contribution to the deformation of the clamping device)

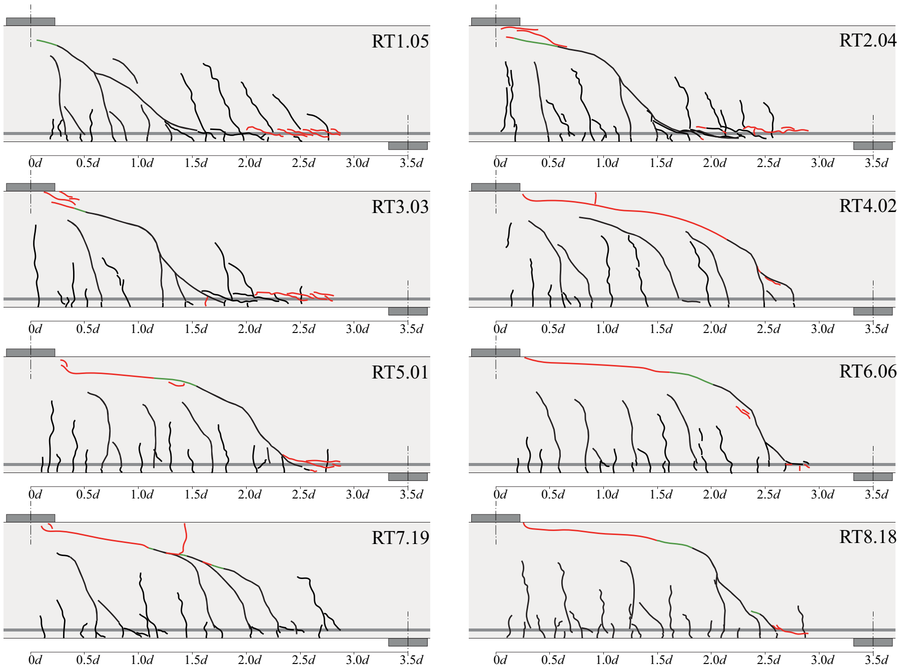

Fig. 5-5 Crack patterns of the slender beam series (a/d = 3.5). Green segments indicate stable post-peak development, red segments indicate unstable development.

Fig. 5-5 and *Fig. 5-6* present the cracking patterns and kinematics observed at different load levels using the DIC technique. The kinematics shown in *Fig. 5-6* refer to the crack opening and the relative tangential displacement between the two lips of a crack and is plotted as a vector for selected load levels (details on how the crack opening and displacements have been calculated can be found in (Campana et al. 2013)). These measurements allow calculating the shear transfer contributions due to residual tensile strength and aggregate interlocking as will be discussed later in this paper. Two trends can clearly be observed depending mostly on the loading rate:

- For high loading rates (specimens RT1.05 – RT3.03), the critical shear crack (CSC) was located close to the mid-support region (between 1.0d – 1.3d from the mid-support axis). It started as a flexural crack and its length increased progressively for higher load levels. When the load was close to 90%–94% of the maximum strength (or first significant drop in the strength for beam RT1.05) other flexural cracks started to merge with it. Yet, an uncracked portion of concrete remained above the CSC and allowed the load to be partly carried by arching action. Failure of specimens RT2.04 and RT3.03 occurred by a crack developing diagonally through the inclined concrete strut near the load introduction region at the mid-support. The response of specimen RT1.05 was a bit different, as it experienced a first drop of load (sudden opening of the CSC probably associated to a loss of aggregate interlock capacity), but it could be reloaded and eventually failed at a higher load by the development of a delamination crack at the level of the flexural reinforcement.

- For both reference and low loading rates (specimens RT4.02 – RT8.18), the critical shear crack (CSC) was located at distances between 2.2d – 2.6d from the mid-support axis. After cracking of the beam, the CSC propagated to a certain length, where it almost stabilized (consistently to the observations of Cavagnis et al. (Cavagnis et al. 2015)). Thereafter, the opening of the CSC increased for higher load levels. Close to the maximum load level, flexural cracks (and sometimes delamination cracks at the level of the flexural reinforcement) merged with the CSC. After reaching the maximum load, the propagation of the CSC continued, becoming eventually unstable.

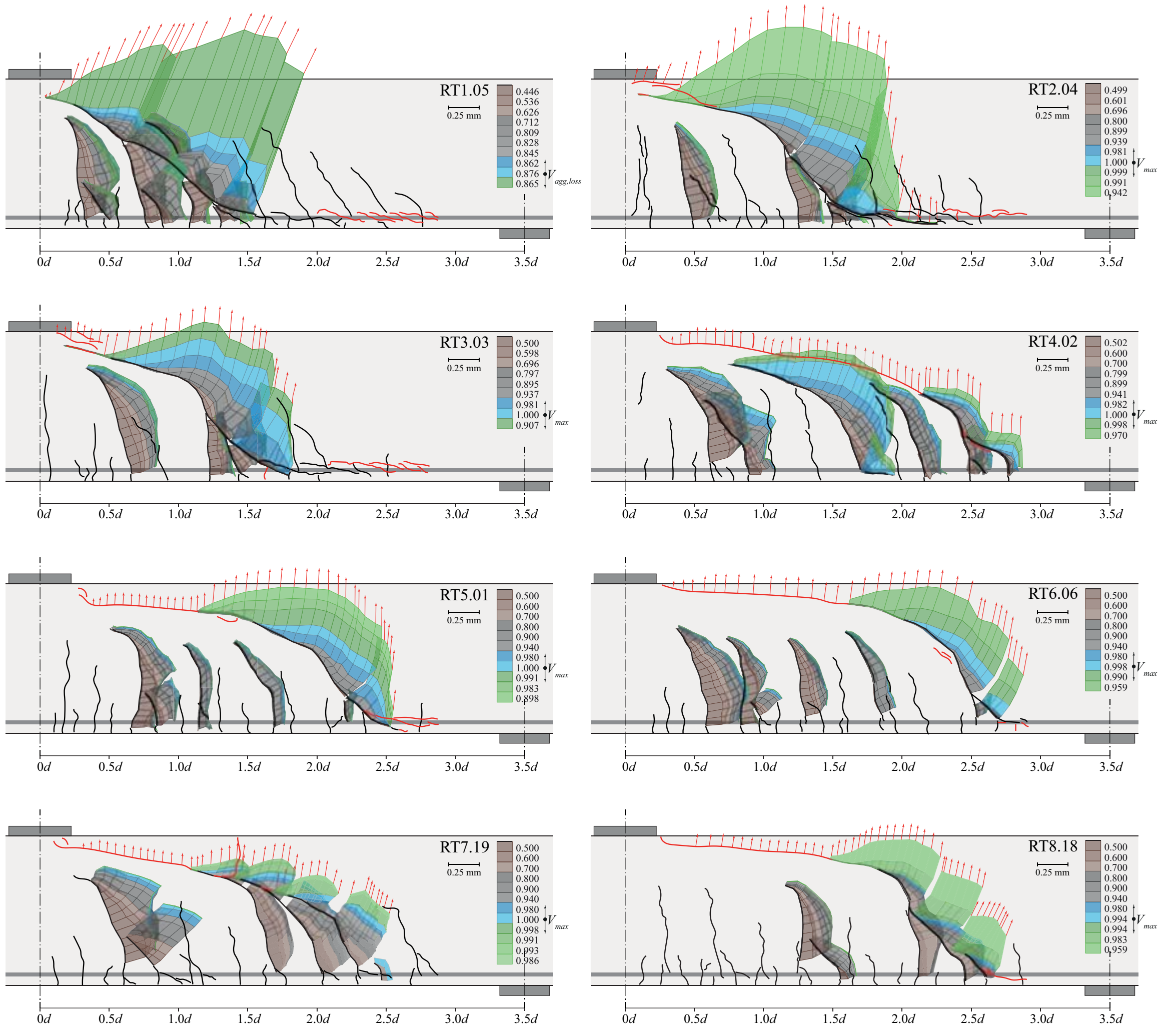

Fig. 5-6 Crack kinematics patterns of the slender beam series (a/d = 3.5). Green vectors indicate stable post-peak displacements, red cracks and red vectors indicate unstable behaviour.

Fig. 5-7 plots the measured horizontal opening of the CSC at two different levels, namely at the level of the flexural reinforcement (sum of the horizontal openings of all cracks in the tributary length l_b, *Fig. 5-7*b) and at the distance of 0.4d from the upper edge (*Fig. 5-7*c). In agreement to the observations of Cavagnis (Cavagnis 2017) an almost linear profile can be observed for the development of the horizontal component of the crack opening, *Fig. 5-7*a. It can be observed that the total horizontal crack widths at the reinforcement level (opening u_A in *Fig. 5-7*b) increase for increasing moments (abscissa in *Fig. 5-7*b) in a similar manner as the experimental observations of Cavagnis (Cavagnis 2017) (refer to dashed straight line in *Fig. 5-7*b). As shown in (Cavagnis 2017), at failure, the horizontal component of the total crack opening (u_A) can be estimated by multiplying the reinforcement strain ($M_A/(A_s \cdot E_s)$) by a tributary length (l_b) that can be estimated as $l_b = d - c$ (where c refers to the thickness of the compression zone).

When the analysis of the crack opening is performed at the level of the reinforcement (*Fig. 5-7*b) the results at failure are mostly dependent on the acting moment, but some influence of the crack opening is also observed with respect to the duration of the test, with low loading rates associated to larger crack openings (dots above the dashed line in *Fig. 5-7*b). With respect to specimen RT1.05 (plotted as a dashed line in *Fig. 5-7*b), the results are not strictly comparable to the others as two potential critical shear cracks developed and merged at the moment of failure. When such analysis of crack openings is performed in the region where aggregate interlock is governing (approximately at 0.4d of the outermost compression fibre (Cavagnis et al. 2018b), see *Fig. 5-7*c), only a clear dependence of the crack width with the level of the acting moment is observed at failure, with no significant influence of the duration of the test. The fact that crack widths are relatively insensitive in this region to the duration of the test can be justified by the fact that two phenomena compensate due to concrete creep: the increase of the sectional curvature (increasing the strains) and the lowering of the neutral axis (decreasing the strains at that region).

With respect to the development of the cracks with time, it is important to note that the opening of the CSC was progressing in a rather stable manner until high levels of load (close to failure). This fact can be clearly observed in *Fig. 5-6*, where the opening until approximately 94% of the failure load is shaded in grey, between 94% and 100% in blue and after reaching the peak load in green (softening phase, unstable failure indicated with vectors in red). As it can be noted in that figure, aggregate interlock was significantly engaged in the last phases of loading (between 94% and 100% of the failure load, blue shaded areas in *Fig. 5-6*) when a rapid but stable opening and sliding of the CSC occurred. As a consequence, the transfer capacity due to aggregate interlock can be considered to have been significantly engaged when the specimen were close to failure, and its contribution shall thus be less influenced by large periods of sustained loading (similar conclusions on non-proportional activation of the various potential shear-transfer actions have been reported and verified elsewhere (Cavagnis et al. 2018b)).

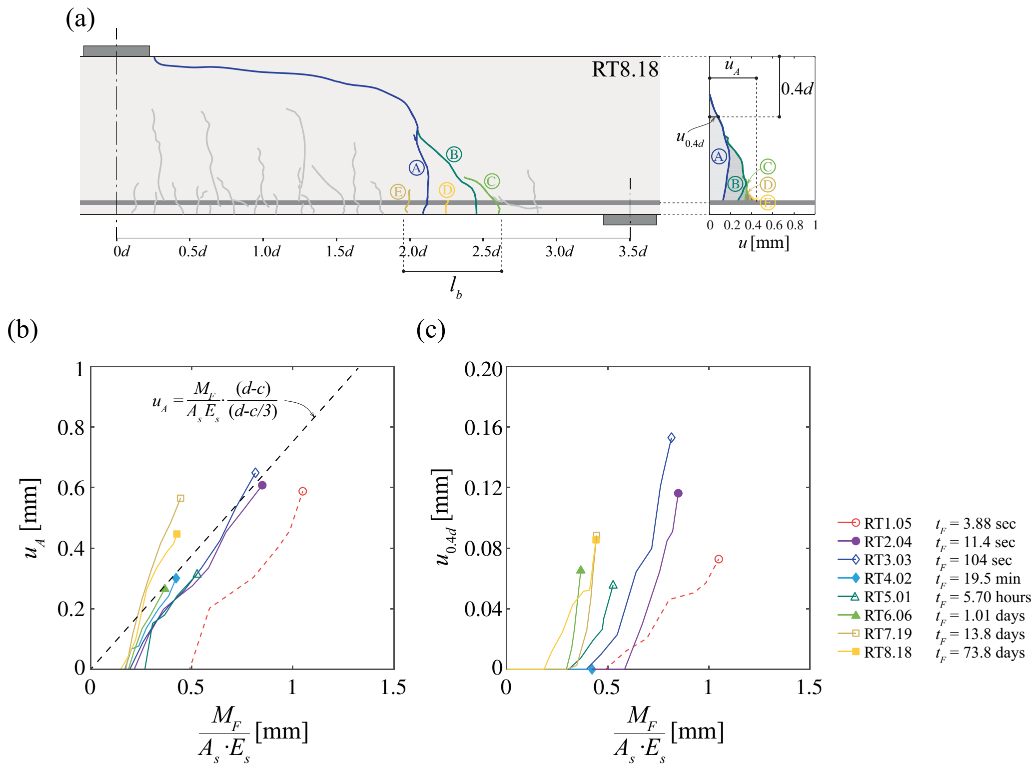

Fig. 5-7 Crack opening in the horizontal axis direction of the beam at different load levels of the slender beam series (a/d = 3.5): (a) definition of horizontal crack opening u_A and $u_{0.4d}$;(b) sum of horizontal openings at the level of the flexural reinforcement over the length l_b; and (c) horizontal openings at 0.4d from the outermost compression fibre

5.4.4 Results for squat beams (a/d = 1.0)

Fig. 5-8 gives an overview of the results of the squat beam series (a/d = 1.0) representing the measured normalized shear force versus the logarithm of time (*Fig. 5-8*a) and the deflection at the load introduction at the end support (*Fig. 5-8*b). With respect to the varying loading rate, particularly for low loading rates, no clear influence can be drawn on the measured shear strength. However, an overall increase of deflection due to creep effects could be observed (almost 100% between the highest (t_F = 9.20 s) and the lowest (t_F = 5.70 days) loading rates).

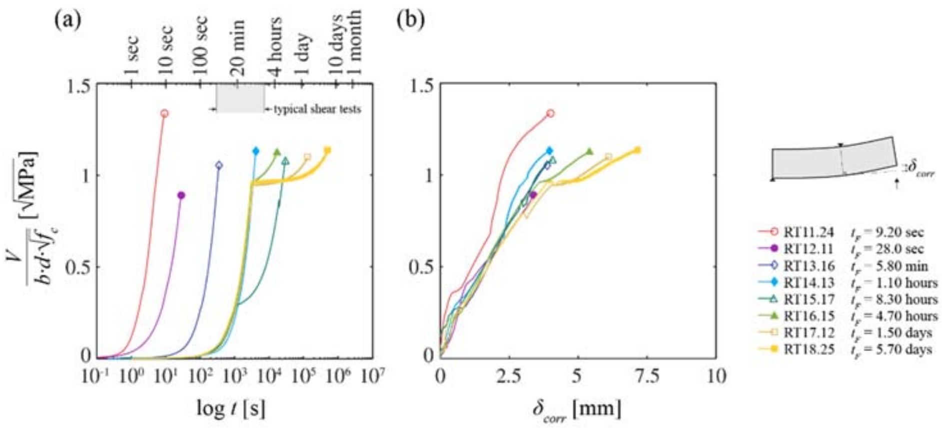

Fig. 5-8 Results of the squat beam series (a/d = 1.0): (a) normalized shear force vs. time in logarithmic scale; and (b) normalized shear force vs. deflection (corrected accounting for the deformation of the test setup and the specimen's rotation at middle section)

Table 5-2 Overview of the results of the squat beam series (t_0 is the age of concrete at load application and t_F is the loading time at failure)

Beam	t_0	$f_c(t_0)$	t_F	$t_{F,250\text{kN}}$	$V_{exp} / (b \cdot d \cdot \sqrt{f_c})$	V_{exp}
	[days]	[MPa]	[-]	[-]	[√MPa]	[kN]
RT11.24	718	37.4	9.20 s	7.40 s	1.34	1'136
RT12.11	697	37.3	28.0 s	19.7 s	0.890	756
RT13.16	613	37.2	5.80 m	4.00 m	1.05	893
RT14.13	683	37.3	1.10 h	47.1 m	1.13	959
RT15.17	696	37.3	8.30 h	7.90 h	1.08	918
RT16.15	640	37.2	4.70 h	4.40 h	1.13	956
RT17.12	646	37.2	1.50 d	1.50 d	1.10	933
RT18.25	750	37.4	5.70 d	5.70 d	1.13	964

Contrary to the slender beams, governed by the development of a single CSC, for the squat beams, a smeared cracking was consistently observed in the upper region of the inclined compression strut carrying shear (where it meets the flexural compression zone). *Fig. 5-9* shows the development of the cracking patterns for three representative specimens (RT11.24, RT12.11 and RT17.12), with several cracks developing in many cases in a quasi-parallel manner (the crack patterns of the other specimens are shown in section 5.9). The direction and intensity of the measured principal compressive strains (blue vectors) and the crack openings (red vectors) are also presented in the same figures. According to the strain measurements, it can be noted that the strains are concentrated in a band between the load introduction plate and the support plate, associated to the development of an inclined compression strut carrying shear by direct strut action.

Fig. 5-10 plots additionally the profiles of the longitudinal strain in the direction of the theoretical compression strut for specimens RT11.24 and RT17.12 (similar plots of other specimens are shown in section 5.9). The base length for the strain calculation was 100 mm (the discontinuities of the plot are due to the presence of localized cracks). An overall increase of the strain in the strut direction can be observed for lower loading rates, mostly pronounced in the zones of the load introduction plates (particularly the one close to the compression zone). It can also be observed that the smeared nature of cracking in the upper part of the strut leads to rather regular strain profiles in this zone. More details can be also seen in *Fig. 5-11*, presenting the integral response of the strut region. With respect to the evolution of the strains in time, it can be observed a gradual activation of the concrete in the region where the compression strut develops, not only in its central part but also at its sides (*Fig. 5-10*). It can be noted that large compressive strains were measured (more than 4‰ for low loading rates) and relatively high transverse strains (more than 8‰) indicating the presence of wide cracks (actual crack openings up to 5 mm).

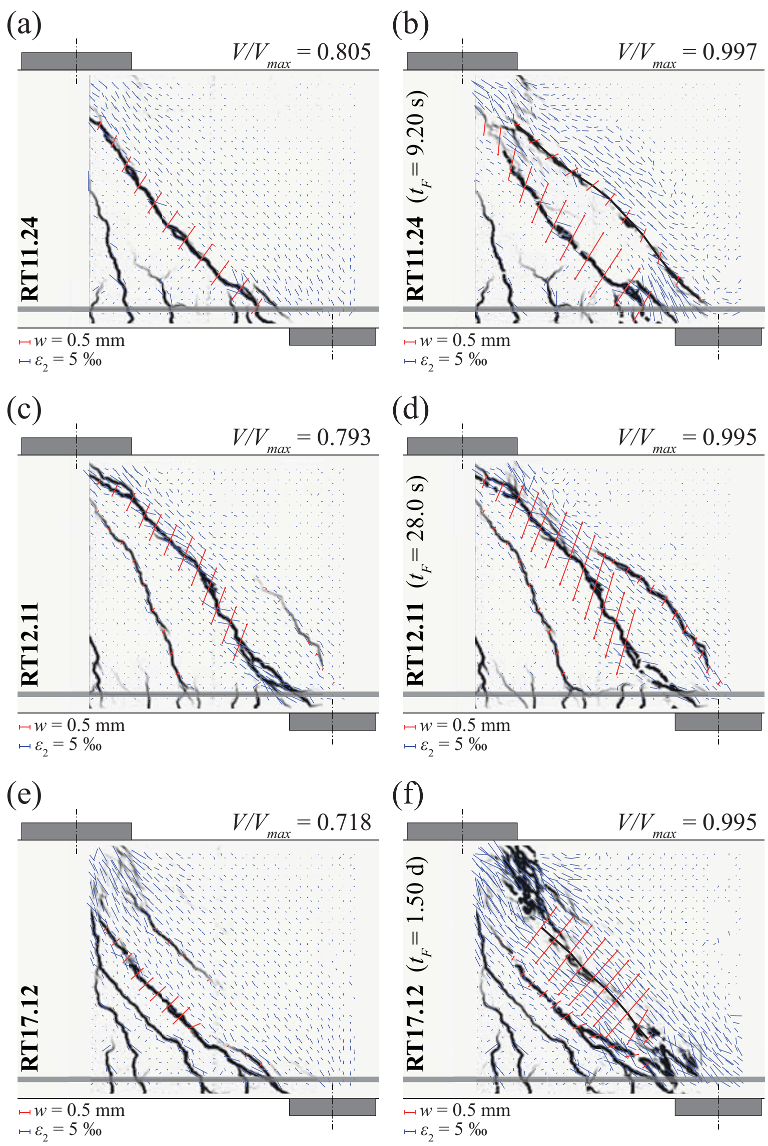

Fig. 5-9 Crack kinematics patterns and principal compressive strain (ε_2) vectors for specimens (a-b) RT11.24; (c-d) RT12.11; and (e-f) RT17.12. The left ones represent a load level of approximately 0.72-0.8$\cdot V_{max}$ and the right ones represent the last available DIC image before V_{max}.

Failure occurred in the specimens by crushing of the concrete in the upper part of the strut. Particularly for the low loading rates, failure was associated to an extensive smeared cracking of this region, see *Fig. 5-9*e-f and section 5.9 (such progression of microcracking under high levels of sustained stress is consistent with the material response observed by Tasevski et al. (Tasevski et al. 2018)). For the high loading rates, it is worth noting that the higher strength observed in specimen RT11.24 can be related to the fact that the inclined compression strut was not disturbed by any wide crack (*Fig. 5-9*a). When the diagonal crack appeared (*Fig. 5-9*b), failure followed instantly. On the contrary, the lower strength in specimen RT12.11 was due to the fact that a diagonal crack, disturbing the strut, already formed at 50% of the maximum strength (*Fig. 5-9*c-d).

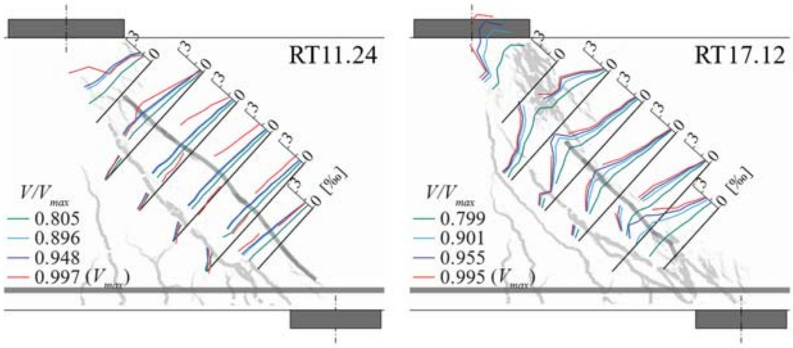

Fig. 5-10 Longitudinal strain profiles in the compression strut for specimens (a) RT11.24; and (b) RT17.12

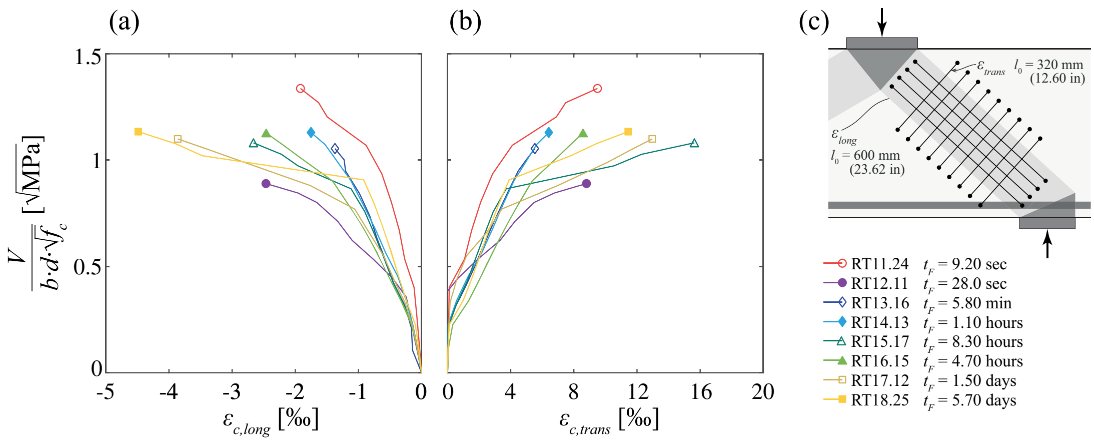

Fig. 5-11 (a) Longitudinal ($\varepsilon_{c,long}$) and (b) transverse ($\varepsilon_{c,trans}$) strain of the direct strut of the squat beams (a/d = 1.0); and (c) base for the measurement of the strains (extraction from DIC data)

5.5 Discussion

5.5.1 Slender beam series

Fig. 5-12 compares the normalized measured shear strength of the slender beam series to the CSCT shear strength prediction ((Muttoni and Fernández Ruiz 2008) where the aggregate size d_g was corrected according to (Cavagnis et al. 2018b) similarly to the plots shown in *Fig. 5-1* for other test series). For higher loading rates, an increase in the strength was rather consistently observed in the tests (in agreement to other results from the scientific literature (Das Adhikary et al. 2014)). For low loading rates, however, no marked decrease of the shear strength compared to typical shear test durations was observed (in agreement to the series previously discussed (Sarkhosh 2014), (Saifullah et al. 2017), see *Fig. 5-1*).

As shown by Campana et al. (Campana et al. 2013), Fernández Ruiz et al. (Fernández Ruiz et al. 2015) and Cavagnis et al. (Cavagnis et al. 2018b), the shear resistance can be calculated as the sum of the contribution of all potential shear transfer actions. For that purpose, the contribution of aggregate interlock can be calculated on the basis of the measured crack opening and sliding together with an aggregate interlock law. In a similar manner, the shear force carried by the residual concrete tensile strength at the crack can be calculated based on of the measured crack opening and considering a constitutive law for the tensile response of concrete. For dowelling action of the flexural reinforcement, the contribution of the reinforcing bars is determined based on the deflected shape of the bars (assumed to behave elastically). Finally, for the shear force carried by the compression zone (arching action), DIC measurements are sufficiently precise to determine the concrete principal strains and to integrate the associated shear stresses (Cavagnis et al. 2018b). The results of such analysis are presented in *Fig. 5-13* where the aforementioned shear transfer actions are calculated consistently with (Cavagnis et al. 2018b) and the total shear strength (addition of all components) is compared to the measured one. As it can be noted, the shape of the CSC and its kinematics significantly influences the various contributions of the shear transfer actions as well as the total shear strength. Some variability is observed with this respect. For instance, the higher strength of specimen RT1.05 (highest loading rate) can be explained by the favourable position of the critical shear crack allowing a significant shear force to be carried by an inclined compression strut. Specimen RT4.02 gave a relatively high strength due to the favourable shape of the crack to the engagement of aggregate interlock. It can also be seen in the analysis that the shear force carried by the compression zone has a relatively larger significance for higher loading rates, while its significance decreases for decreasing loading rates (where aggregate interlock becomes governing).

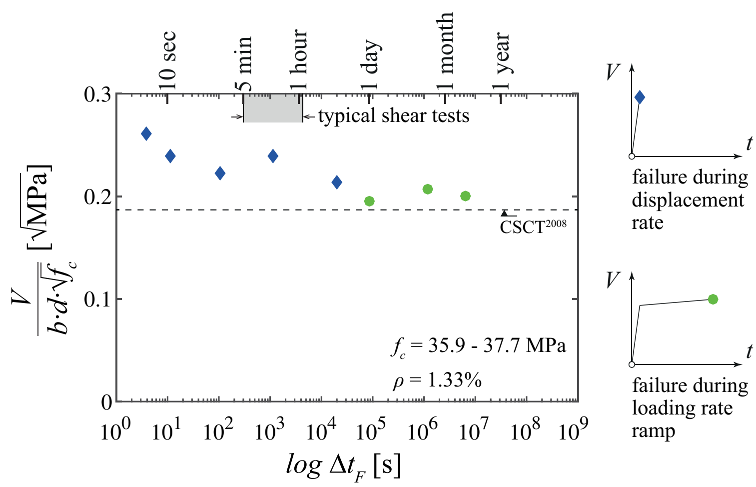

Fig. 5-12 Comparison of the slender beam series (a/d = 3.5) with CSCT^{2008} (Muttoni and Fernández Ruiz 2008)

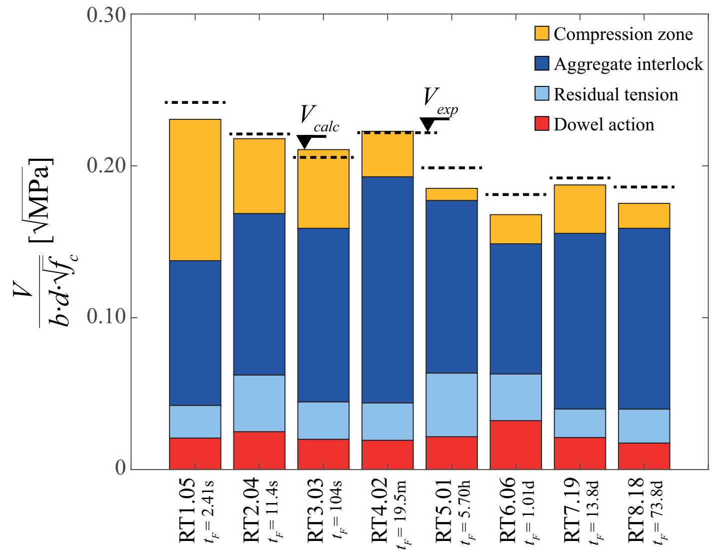

Fig. 5-13 Contribution of different shear transfer actions at maximum load for the slender beam series (a/d = 3.5)

Overall, a fairly good estimate of the shear strength is obtained following this approach (average measured-to-calculated strength of 1.03 with a coefficient of variation of only 3.6%). The more favourable shape of the crack developed for high loading rates allowed for larger capacities than for specimens loaded at reference or low loading rates.

It is important to note that the resistance of the shear transfer actions does not seem to need any correction with respect to the potential softening due to sustained loading effects on the material behaviour (all members have been analysed using the same constitutive laws and without accounting for potential short- or long-term influences). This observation can be considered consistent for the aggregate interlock contribution (dominant shear-transfer action in the slender beams subjected to low loading rates). This is justified by the fact that a comparable shape of the CSC developed (*Fig. 5-6*), with similar crack openings in the critical region for aggregate interlock transfer (*Fig. 5-7*c), and by the fact that the aggregate interlocking was significantly engaged in a relatively rapid manner when the specimen were close to failure (blue areas in *Fig. 5-6*, refer to section 3.3). These aspects allow to consider that the contribution of aggregate interlock was little sensitive to low loading rates (similar crack shape, opening and significant engagement of the shear-transfer action in a rapid manner close to failure). A similar consideration can also be performed for the residual tension contribution and dowelling action (associated to the rapid opening of the CSC close to failure). With respect to the contribution of the compression zone, this action can potentially be more influenced by high levels of sustained load (Tasevski et al. 2018). However, its overall contribution to the shear strength was limited for low loading rates and slender members and with the possibility to develop stress redistributions (this aspect is commented in detail in the next section with reference to the arching action in squat members).

5.5.2 Squat beam series

In the squat beam series, a larger scatter in terms of strength was observed for higher loading rates. For reference or low loading rates, almost no variation on the shear strength was observed. This fact is shown in *Fig. 5-14*, where the measured shear strength is compared to that of a rigid-plastic stress field (lower bound according to limit analysis) (Muttoni et al. 2015) considering an effective concrete strength $f_{ce} = 0.75 \cdot f_c$ where a constant efficiency factor $\nu = 0.75$ for the concrete strut is assumed (accounting for concrete brittleness and influence of transverse cracking (Fernández Ruiz and Muttoni 2008)) and assuming that the nodal zones are not governing. It can be noted that this value of the efficiency factor is higher than the one suggested by ACI 318-14 (ACI 2014) for the inclined strut ($\nu = 0.85 \cdot \beta_s = 0.51$). This topic is discussed later in this section.

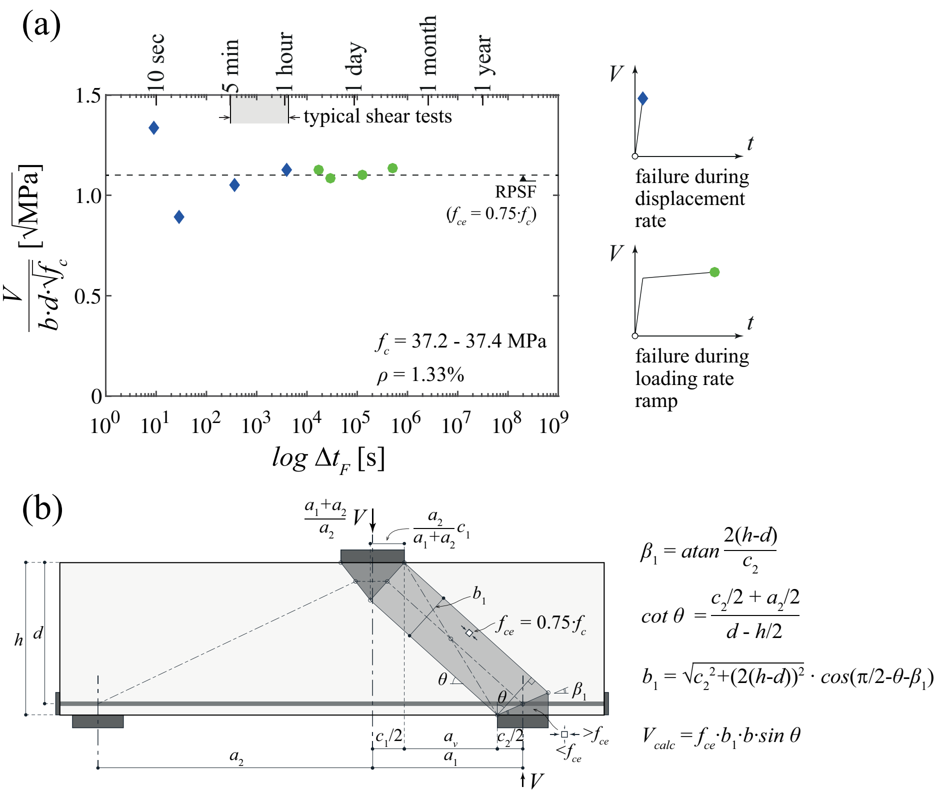

Fig. 5-14 (a) Comparison of the squat beam series (a/d = 1.0) with a rigid-plastic stress field solution (effective concrete strength $f_{ce} = 0.75 \cdot f_c$); and (b) detail of the stress field

With respect to this relatively insensitive response to the duration of the load, one could expect that the concrete strut may reduce its strength when subjected to high levels of sustained stress during long periods (Rüsch 1960; Tasevski et al. 2018). This phenomenon, however, appeared to be compensated by the potential confinement of the strut by the surrounding concrete (restraining the transversal deformation of the strut) and the influence of the surrounding concrete due to the activation of additional concrete at the sides of the strut (increase of the area of the strut). The development of a confinement of the strut by the surrounding concrete can be supported by the measurements of *Fig. 5-15*. The crack below the compression strut (crack characterized by point A) opens at early stages of loading (*Fig. 5-15*a,c, blue lines), but it closes completely or partly for higher load levels after the crack in the inclined strut develops (crack with point B in *Fig. 5-15*b,c, refer to green lines). Such closure was particularly notable at its upper part where the crack lips were again in contact (thus allowing to transfer compressive stresses). As a consequence of this crack closure and potential confining stresses, larger portions of concrete could be activated and the opening of crack B was controlled. This is shown in a schematic manner in *Fig. 5-15*d and can also be observed in *Fig. 5-9* and section 5.9. These effects allowed for an enhanced performance of the strut, thus leading to relatively high calculated values of the efficiency factor of the strut. It can additionally be noted that the strut cannot be confined in the upper region since no longitudinal reinforcement is available to ensure the equilibrium of the deviation forces. A similar effect has been demonstrated for the case of footings with concentrated loading where the reinforcement in the compression zone can increase the load-carrying capacity (Simões et al. 2016).

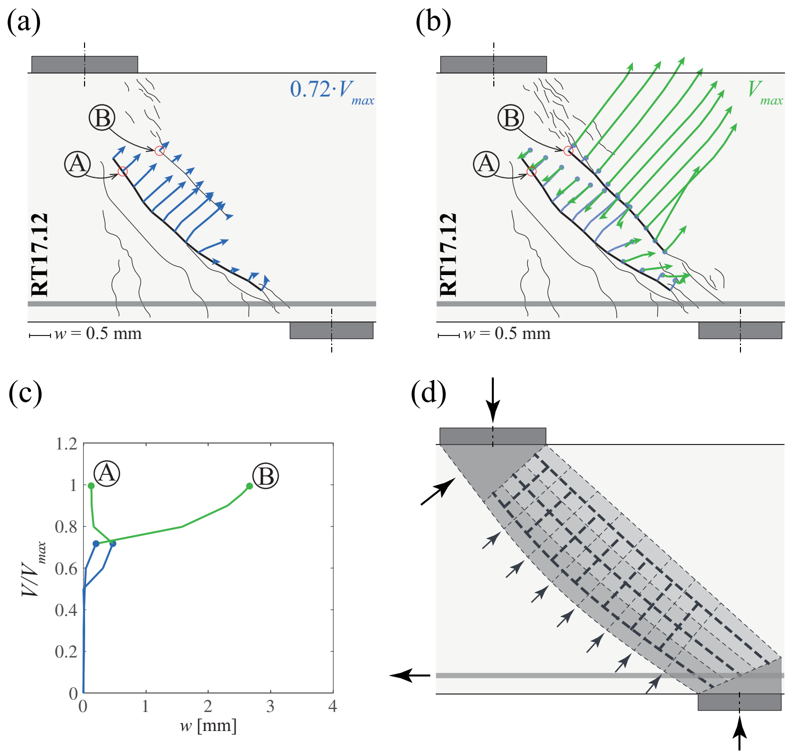

Fig. 5-15 Crack kinematics for specimen RT17.12: (a) Crack opening at $0.72 \cdot V_{max}$; (b) crack closure at V_{max} for lower crack; (c) plot of crack evolution at locations A and B; and (d) potential confinement of the strut.

5.5.3 Practical design recommendations

As previously discussed, codes of practice show no consensus on the consideration of the effect of long-term actions on the shear strength. On the basis of the test results and the considerations discussed in this manuscript, there seems to be no need to consider a strength reduction for shear critical members without transverse reinforcement subjected to high levels of sustained load with respect to members loaded at a reference loading rate (failure in 5 minutes - 2 hours). This conclusion shall be understood valid for the duration of the loading, member slenderness and material property ranges investigated in this paper and is supported on the following considerations:

- For slender beams, the shape of the CSC seems to be governing for the strength. For high loading rates (higher than the reference one), a more favourable shape of the CSC may develop, but no significant modification in the shape of the crack or shear strength is observed for lower loading rates compared to typical shear tests. This limited sensitivity of the shear strength for low loading rates can be justified by the similar shape and openings of the CSC and by the relatively rapid (yet stable) opening and sliding of the CSC close to failure.

- For squat beams, the capacity of stress redistribution and potential confinement by the concrete surrounding the direct strut region seems to compensate for the potential decrease of material strength under sustained loading during long periods of time.

5.6 Conclusions

This paper presents the results of an experimental investigation on the influence of the duration of loading on the shear strength of members without shear reinforcement. Its main conclusions are presented below:

1. The tests presented in this paper do not show any marked decrease on the shear strength for higher durations of application of the load or for low loading rates compared to typical shear tests.

2. For loading rates higher than typical shear tests (associated to failures after some seconds), a relatively consistent increase on the strength can be observed for slender specimens. An analysis of the shear transfer actions shows that this can be justified by a more favourable geometry of the critical shear crack in case of high loading rates. The results are not so clear for squat members but indicate a similar response.

3. Despite the fact that for the pure material response a decrease on the strength has been largely documented and verified, for members in shear, it seems that there is no need for a reduction in the shear resistance for high levels of sustained loads during long periods of time.

4. For squat members, this conclusion can be justified by the potential redistribution and confinement capacity of the concrete surrounding the strut carrying shear.

5. For slender members, the limited sensitivity of the shear strength to low loading rates can be justified by the similar shape and openings of the critical shear crack (CSC) and by the relatively rapid (yet stable) opening and sliding of the CSC close to failure. This allowed to significantly engage aggregate interlocking (the dominant shear-transfer action in these members) in a relatively rapid manner (this contribution being thus little influenced by the total duration of the loading process).

These observations are yet based on the currently limited available experimental data in the scientific literature and a wider set of experimental results would be required to ensure the general applicability of these conclusions.

5.7 Notation

Term	**Signification**
a	shear span (distance between load axis and support axis)
A_s	area of flexural reinforcement
b	width of the member
d	effective depth
d_g	aggregate size
E_s	modulus of elasticity of reinforcing steel
f_c	uniaxial compressive strength of concrete [MPa]
$f_c\ (t_0)$	reference compressive strength obtained at a strain rate of 0.02 ‰·s^{-1} and at an age t_0 [MPa]
$f_{c,28}$	reference compressive strength obtained at an age of 28 days [MPa]
f_y	yield strength of reinforcement (corresponding to the 0.2% proof stress)
h	height of the member (or cylinder)
l_b	length of the region of the beam contributing to the critical shear crack opening
M_F	bending moment at the section of the tip of the shear crack
RH	relative humidity
T	temperature
t	time
t_0	concrete age at loading [days]
t_F	time to failure since beginning of tests
$t_{F,100kN}$	time to failure after reaching a load of 100 kN
t_s	concrete age at the beginning of drying [days]
u_A	measured horizontal opening of the critical shear crack at the flexural reinforcement level
$u_{0.4d}$	measured horizontal opening of the critical shear crack at distance of 0.4d from the compression face
V	shear force
V_{calc}	calulated shear strength
V_{exp}	experimentally measured shear strength
V_{max}	maximum measured shear force
w	crack opening perpendicular to the crack surface
δ	deflection at the load introduction (at the edge support)
δ_{corr}	corrected deflection after considering rigid body movements
ε_{cs}	shrinkage strain
$\dot{\varepsilon}_{ref}$	reference strain rate for uniaxial compressive strength testing
$\varepsilon_{c,trans}$	transverse strain
$\varepsilon_{c,long}$	longitudinal strain
ε_2	second principal strain
$\O$	concrete cylinder diameter

φ_{lin}	linear creep coefficient
ν	efficiency factor for the concrete strut
ρ	flexural reinforcement ratio
σ_c	concrete stress

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5.9 Appendix 1

This Appendix presents the crack kinematics patterns and principal compressive strain (ε_2) vectors of specimens RT13.16 - RT18.25 of the squat beam series (*Fig. 5-16* - *Fig. 5-18*) as well as the longitudinal strain profiles in the compression strut for all specimens of the squat beam series (*Fig. 5-19*).

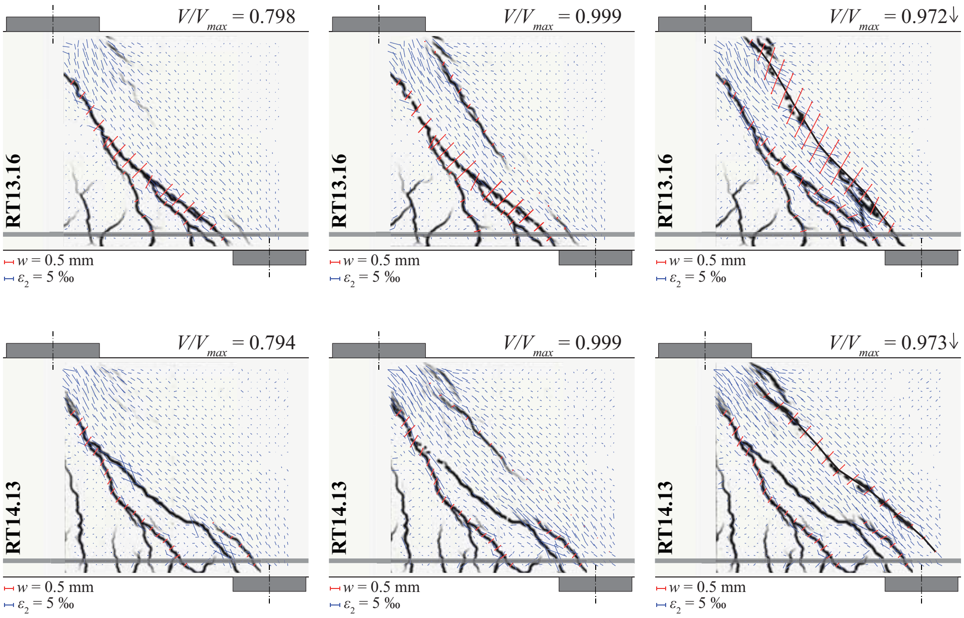

Fig. 5-16 Crack kinematics patterns and principal compressive strain (ε_2) vectors for specimens RT13.16 and RT14.13 at three load stages: approx. $0.8 \cdot V_{max}$, approx. Vmax and instants before failure

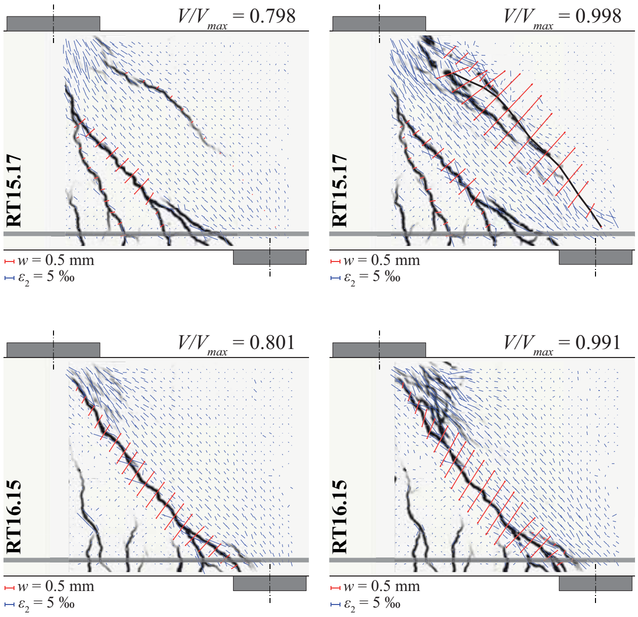

Fig. 5-17 Crack kinematics patterns and principal compressive strain (ε_2) vectors for specimens RT15.17 and RT16.15 at two load stages: approx. 0.8· V_{max} and approx. V_{max}

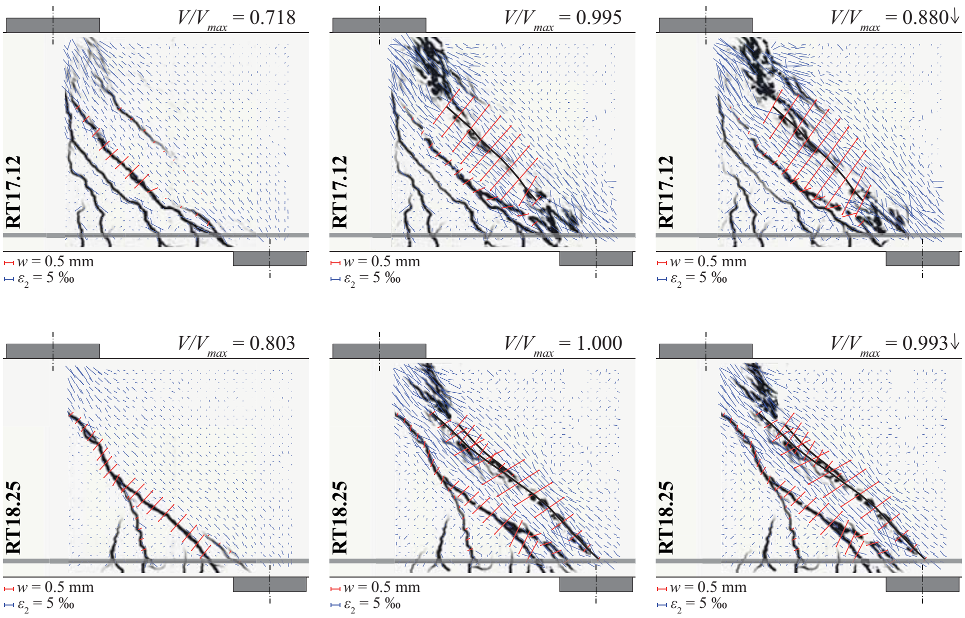

Fig. 5-18 Crack kinematics patterns and principal compressive strain (ε_2) vectors for specimens RT17.12 and RT18.25 at three load stages: approx. 0.72-0.8· V_{max}, approx. V_{max} and instants before failure

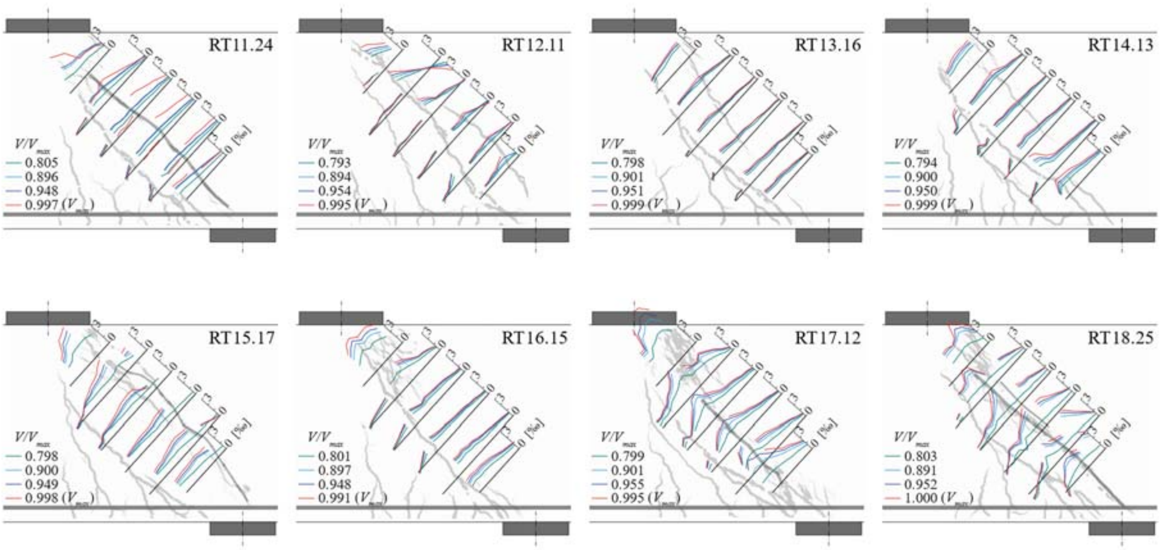

Fig. 5-19 Longitudinal strain profiles in the compression strut of the squat beam series (a/d = 1.0)

Chapter 6 Conclusions and outlook

6.1 Introduction

The investigations performed in this thesis contribute to better understand the phenomenon of time-dependent strength of concrete under various loading patterns ranging from rapid to sustained actions. This phenomenon is investigated on both uniaxial compression specimens and shear-critical reinforced concrete members.

With the purpose of investigating the development of linear and nonlinear creep strains potentially leading to failure under high levels of load, new experimental data is presented on loading patterns with variable strain and stress rates. Based on an improved mechanical model, which has been validated by these tests and additional tests from scientific literature, it has been shown that the strength reduction, which can reach approximately 20%, is more severe under a constant sustained load than in the case of loads increased with a slow rate. In addition, it could be observed that the age of loading plays a major role on the strength reduction. The strength reduction occurs rapidly in case of loading of concrete at young age, but it is very slow for old concretes. All these considerations show the importance of the strength increase due to continued cement hydration and the age of assessing the reference concrete strength. In addition, both the mechanical model and the conducted tests show that the detrimental effect of sustained loading and of slow loading rates on the compressive strength is associated with a larger deformation capacity at failure. This effect can be very beneficial and can potentially compensate the detrimental effect described above, allowing for a better redistribution of internal forces, reducing stress concentrations in concrete and allowing for an increase of the reinforcement activation in compression before concrete failure.

For a detailed verification of structures with a complex loading history, the cumulative damage estimation approach of Palmgren-Miner has been adapted to account for time-dependent effects, leading to consistent results. This has allowed to study several practical cases such as variation of constant stress rates or variation of constant strain rates, constant slow stress rate after a rapid initial stress increase (typically accounting for the stress increase during construction when additional permanent loads are added over a couple of months after a rapid construction), rapid stress increase after a period of sustained loading (simulating rapid variable actions acting in addition to permanent loads) and the influence of the duration of variable actions.

For practical design considerations, it must be remembered that the effect of a slow loading rate is partially accounted for in code provisions in an implicit manner, because design equations have been calibrated on laboratory tests with typical durations between approximately 20 minutes and a couple of hours. This also allows reducing the period during which the detrimental sustained loading effect is not fully compensated by the favourable strength increase due to continued cement hydration.

To verify whether a shear strength reduction due to sustained loading is necessary also for members without shear reinforcement, two test series with varying loading rates were conducted on slender and squat members. Both series, as well as a comparison with other test series found in the scientific literature, show that there is no marked decrease of the shear strength for longer durations of application of the load or for lower loading rates compared to typical shear tests. This can be explained by the favourable effect of the stress redistribution due to the nonlinear creep associated with sustained loading or slow loading rates. Furthermore, loading rates higher than typical shear tests show a consistent increase of the strength, possibly governed by the favourable geometry of cracking patterns. Based on the present work, for the assessment of shear critical existing structures, it can be assumed that a strength reduction factor for the effect of sustained loads is not necessary.

6.2 Conclusions

6.2.1 Uniaxial compressive strength

Based on an experimental campaign of uniaxial compression with varying strain and stress rates, following conclusions on the time-dependent behaviour of concrete under high levels of sustained load can be drawn:

1. The compressive strength of concrete reduces due to sustained loading or low strain or stress rates. This is a detrimental effect in members subjected to high stresses.

2. The corresponding strain at failure increases. This may be an instrumental positive effect for statically redundant systems, where redistributions of stresses are possible and may increasingly activate compression reinforcement.

3. Failure under sustained load is governed by the inelastic strain capacity of concrete. When the developed inelastic strain of concrete equals the inelastic strain capacity, failure occurs by progressive coalescence of cracks.

4. The inelastic strain capacity can be estimated as the difference between the instantaneous post- and pre-peak strains for a given stress level. This allows defining a failure criterion for the available inelastic strain capacity.

Furthermore, the development of linear and nonlinear creep strains potentially leading to failure under high levels of load has been investigated in an analytical manner, by applying existing creep models and plausible assumptions inspired from the mechanical behaviour of concrete. This analytical work has resulted with a consistent mechanical model, which exploration allows to draw some further conclusions:

5. The affinity assumption between linear and nonlinear creep strains allows to formulate a simple and efficient relation to estimate the development of inelastic strains in concrete for a given loading history. It accounts in an explicit manner for the development of tertiary creep strains.

6. Failure under long-term loading patterns can be analytically calculated as the intersection between the relationship defining the development of inelastic strains for a given loading pattern and the failure criterion defined by the inelastic strain capacity of concrete. The

strength and the associated strain can be consistently estimated with this approach for different loading patterns, as validated by test results on both sustained load and stress and strain rates.

7. A further exploration of the theoretical model shows that, if concrete is subjected to constant sustained stress, failure occurs at lower stress levels and quicker than in the case of stress or strain rates. This naturally results from the fact that in the former case, concrete is subjected to higher stress levels since the beginning of the loading process.

8. With the help of the theoretical model, it could also be confirmed that the increase of concrete strength with time is an instrumental phenomenon for the compensation of material damage. For early ages of loading, failure under sustained load can only occur after some hours or days, while for concretes loaded at older ages, failures may occur several years after loading (no more increase of strength due to cement hydration).

6.2.2 Shear strength of reinforced concrete members

Based on an experimental campaign on shear-critical slender and squat beams without transverse reinforcement subjected to varying loading rates, following conclusions on the time-dependent shear strength of reinforced concrete members can be drawn:

1. The performed tests do not show any marked decrease on the shear strength for higher durations of application of the load or for low loading rates compared to typical shear tests.

2. For loading rates higher than typical shear tests (associated with failures after some seconds), a relatively consistent increase on the strength can be observed for slender specimens. An analysis of the shear transfer actions shows that this can be justified by a more favourable geometry of the critical shear crack in case of high loading rates. The results are not so clear for squat members but indicate a similar response.

3. For squat members, this conclusion can be justified by the potential redistribution and confinement capacity of the concrete surrounding the strut carrying shear.

4. For slender members, the limited sensitivity of the shear strength to low loading rates can be justified by the similar shape and openings of the critical shear crack (CSC) and by the relatively rapid (yet stable) opening and sliding of the CSC close to failure. This allowed to significantly engage aggregate interlocking (the dominant shear-transfer action in these members) in a relatively rapid manner (this contribution being thus little influenced by the total duration of the loading process).

6.2.3 Design code aspects

Based on the experimental and analytical work of this thesis, a simplified code-like approach is proposed for engineering practice. Some important aspects are:

1. The phenomenon of concrete creep shall be accounted for both at Serviceability Limit States (SLS) and at Ultimate Limit States (ULS).

2. The failure of concrete under sustained load can be described by simple code-like expressions. Combining these expressions with the Palmgren-Miner's rule provides a practical manner to account for damage accumulation and to estimate the failure load and time for different loading patterns.

3. The beneficial effect of continued cement hydration usually compensates for the detrimental effect of sustained loading. This holds true provided that the concrete strength is determined at 28 days and the ULS occurs after a sufficiently long period of time. For existing structures, where the concrete strength is determined at an age later than 28 days, the reduction of concrete strength accounting for high sustained long-term loading is justified.

4. For a given level of total stress, a combination of permanent actions and a rapid variable action is less detrimental for the concrete compressive strength than a full permanent action. Simple, code-like expressions could be developed to describe this phenomenon, resulting in a linear interpolation between the response of a member failing under constant sustained load and the case where permanent load effects can be neglected. A similar approach can be followed for cases where the variable action is maintained over a period of time.

5. According to the simulation of the simplified load histories, it can be assumed that for loading durations up to several hours, variable loads may be considered as sufficiently rapid to neglect their influence on the compressive strength.

6. Despite the fact that for the pure material response a decrease on the strength has been largely documented and verified, for members in shear, it seems that there is no need for a reduction in the shear resistance for high levels of sustained loads during long periods of time.

6.3 Future works and outlook

There are still some open questions on the time-dependent strength of concrete and concrete members. With respect to the strength of compression members and compression zones is structures, following topics could be addressed:

- The influence of different types and magnitudes of confinement on the nonlinear creep response and time-dependent strength of concrete. This topic would require new experimental evidence with the aim to extend existing analytical models for the effects of confinement. Two main aspects could be considered:

    a. Time-dependent strength of concrete in bi- and triaxial conditions (active confinement), in order to develop an analytical failure criterion for application in mechanical models (for instance in shear and punching strength models).

b. Effects of passive confinement given by the presence of reinforcement or prestressing in order to address the phenomenon in typical elements such as compression members and compression zones in bending confined by stirrups, zones with transverse reinforcement carrying shear, confined zones of load or prestress force introduction etc.

- The contribution of compression reinforcement on the response of compression members and compression zones in bending under high sustained load. More precisely, the activation of compression reinforcement with nonlinear creep could be studied as a function of the compressive reinforcement ratio, in order to understand if there is a minimum reinforcement ratio at which the beneficial effects of nonlinear creep prevail the detrimental effects.

With respect to the strength of shear-critical members, following topics could be addressed:

- The time-dependent strength of shear critical members could be studied for a broader range of shear span to effective depth ratios and flexural reinforcement ratios.

- The influence of the loading rate and stress sustain on the different shear transfer actions (mechanisms), for instance on the residual tensile strength, aggregate interlock and dowelling action could be studied. This aspect would require new experimental evidence in order to improve the existing constitutive laws that describe these mechanisms for the time-dependent effect of load. This approach could help to further explore the influence of sustained loads on shear critical members under different loading and boundary conditions.

- The influence of high sustained loads on the punching strength of flat slabs and footings. This topic would require new experimental evidence in order to clarify the influence of time-dependent loads on the punching strength.

The coverage of these questions may help to reach a consensus of the practical design rules of different codes of practice.

Several other topics beyond the scope of interest of this thesis could be of interest for the broader concrete research community:

- The phenomenon of time-dependent strength and the influence of sustained and rapid loads on the behaviour and strength of different types of concrete, including high strength concrete, ultra-high performance fiber reinforced concrete (UHPFRC) as well as other cement-based materials for special application.
- The influence of high sustained loads under severe environmental conditions. It is known that both frost and thaw cycles and very high temperatures can cause severe damage in concrete, which in addition to sustained load damage, may aggravate the state of a structure and result with highly detrimental effects.
- The influence of high sustained loads on severely damaged concrete due to alkali-aggregate reaction or sulfate attack.

Appendix A1 Inelastic strain development in experimental series

Section 3.4.5 discusses the developed nonlinear (inelastic) creep strain at failure in the tests of the performed experimental campaign. This appendix complements section 3.4.5 with detailed plots of the inelastic strain development and corresponding inelastic strain capacity. The plots of the 1st test series with constant strain rate are presented in *Fig. A1-1* and the ones of the 2nd test series with constant stress rate are presented in *Fig. A1-2*. The gray shade represents the scatter interval of the measured inelastic strain capacity.

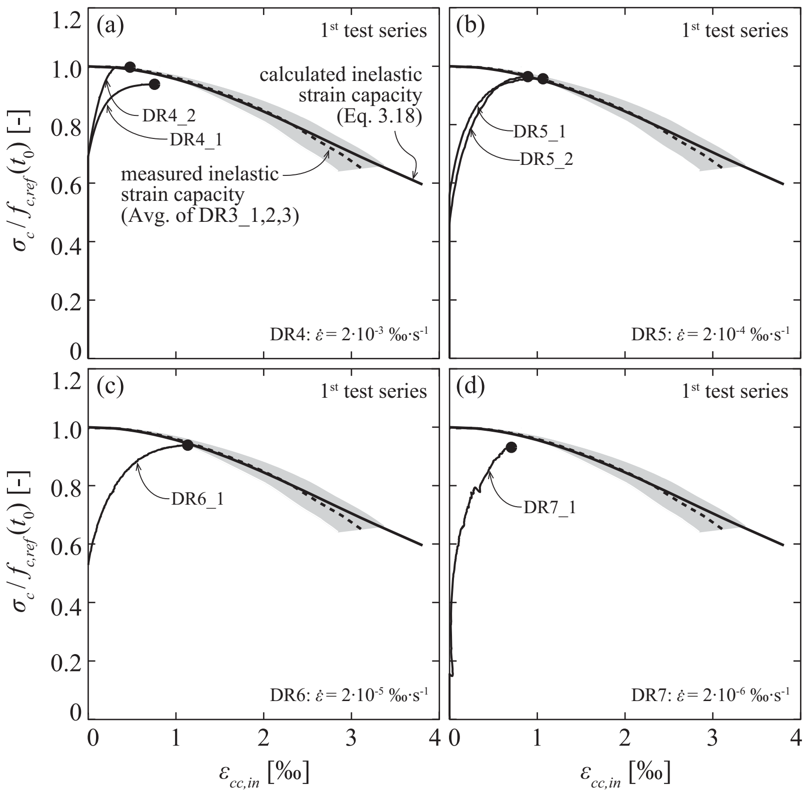

Fig. A1-1 Inelastic strains and corresponding inelastic strain capacity for the 1st test series with constant strain rate

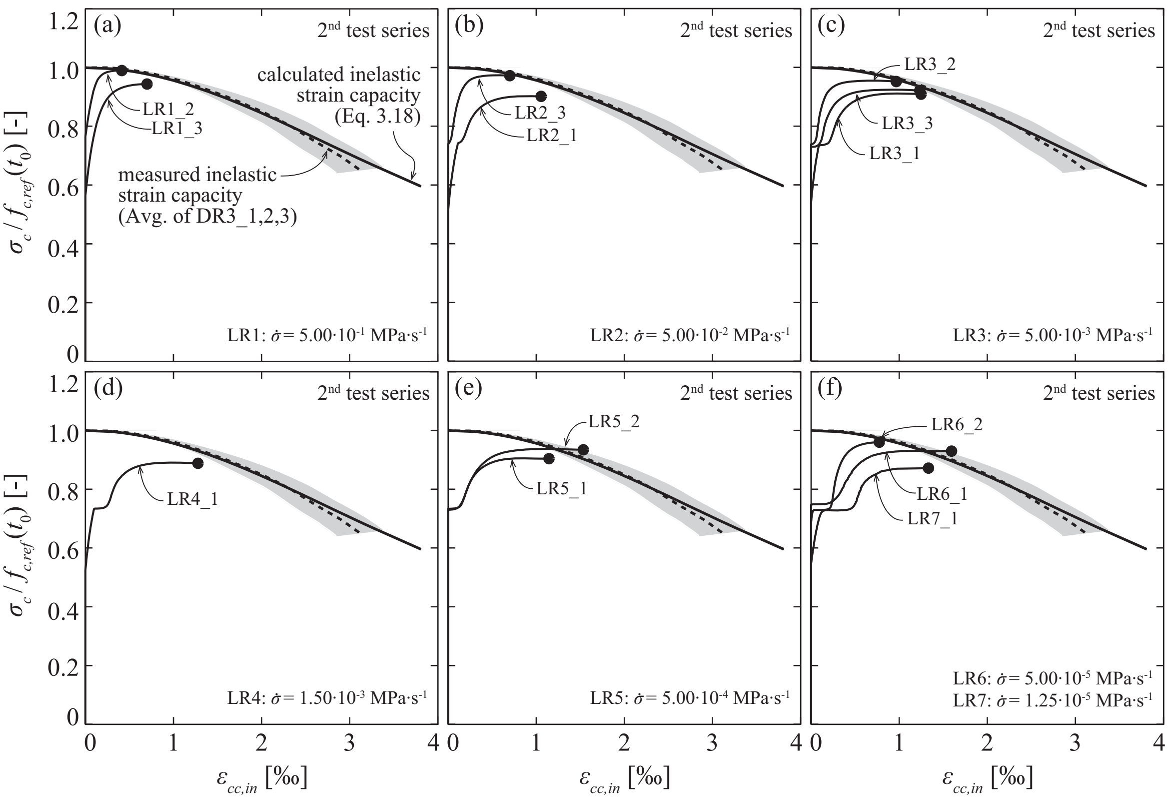

Fig. A1-2 Inelastic strains and corresponding inelastic strain capacity for the 2nd test series with constant stress rate

Appendix A2 Extension of parametric analyses

This appendix has the purpose to extend the parametric analysis performed in section 3.7. It completes the results presented in *Fig. 3-17* and *Fig. 3-18*, by analysing the loading cases of constant sustained stress and constant stress rate for different loading ages. The resulting strength is represented by normalization to the reference strength at three different ages, namely the age of 28 days (*Fig. A2-1*), the age of loading (for each curve, *Fig. A2-2*) and the age at failure (for each point, *Fig. A2-3*).

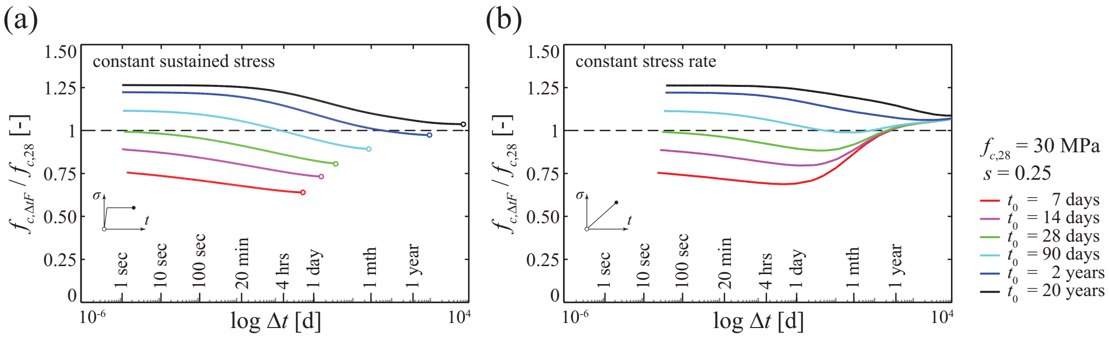

Fig. A2-1 Parametric analysis for different loading ages of the effect of (a) constant sustained stress; and (b) constant stress rate (strength increase with concrete age considered, strength normalized to the reference strength determined at the age of 28 days)

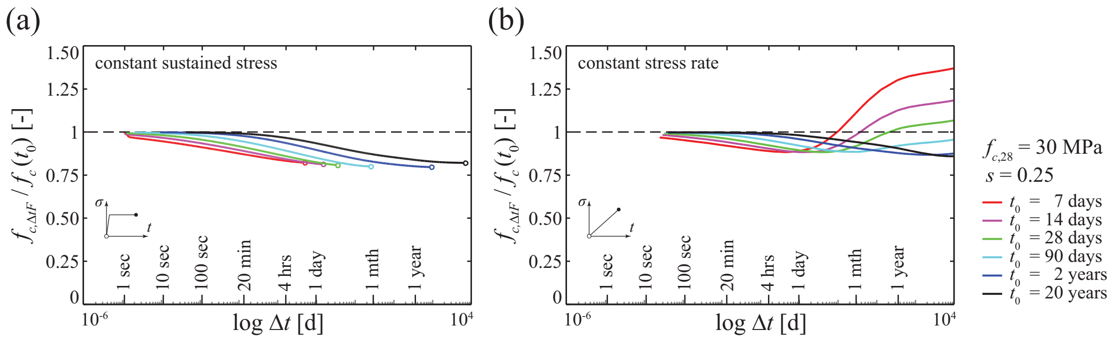

Fig. A2-2 Parametric analysis for different loading ages of the effect of (a) constant sustained stress; and (b) constant stress rate (strength increase with concrete age considered, strength normalized to the reference strength determined at the age of load application)

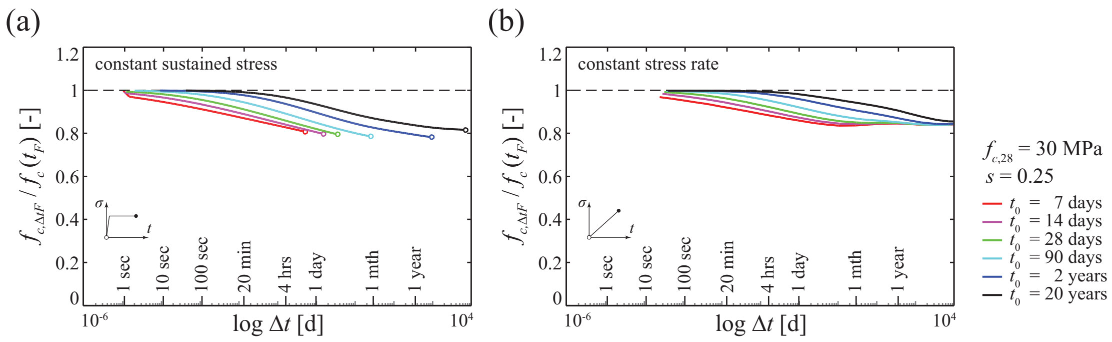

Fig. A2-3 Parametric analysis for different loading ages of the effect of (a) constant sustained stress; and (b) constant stress rate (strength increase with concrete age considered, strength normalized to the reference strength determined at the age at failure)

Appendix A3 Mechanical analogy of inelastic strain development due to microcracking

For sustained load levels lower than $\sigma/f_c \approx 0.4$, the creep development is assumed to be only correlated to the viscoelastic properties of the cement paste and there is quasi no development of damage in the material (no microcracks present). At load levels $\sigma/f_c > 0.4$ (behaviour illustrated in *Fig. A3-1*a), the initiation of microcracks in the material starts. A part of it occurs during the load application (stage AB in *Fig. A3-1*a) and another part during the stage of sustained loading (from point B in *Fig. A3-1a* on). The creep process initiates with a primary creep phase immediately after the load application (stage BC in *Fig. A3-1*a), followed by a secondary creep phase (stage CD in *Fig. A3-1*a). When such high loads are sustained over large periods of time, this latter phase is characterised by a constant creep rate, justified by a stable propagation of microcracks. At load levels higher than $\sigma/f_c \approx 0.75$–0.8, the microcrack propagation may possibly become unstable as the coalescence of microcracks starts. This describes the onset of the tertiary creep phase (stage DE in *Fig. A3-1*a). This phase is characterized with an increasing rate of time-dependent deformations resulting in a final process of progressive crack coalescence (stage EF in *Fig. A3-1*a) leading to failure.

To describe the behaviour under sustained loading, many authors (refer for instance to (El-Kashif and Maekawa 2004) and (Fernández Ruiz et al. 2007)) used the analogy of a coupled elasto-plastic and damage model and a coupled elasto-visco-plastic and damage model respectively. The base component of the model is a chain of elastic spring, plastic slider and viscous damper (dashpot) connected in series. The model is constituted of several such chains connected paralelly. The viscous damper element in the mechanical analogy of (Fernández Ruiz et al. 2007) represents the portion of viscoelastic strain, i.e. time-dependent strain not associated with micro-cracking, whereas the viscoplastic strain is represented together with the time-independent plastic strain by the plastic slider elements. Damage, which is entirely associated with cracking, is represented by losing entire single chains after their maximum capacity have been reached. However, if all the parallel connected chains have the same mechanical properties, they should all reach their capacity in the same moment.

Tertiary creep of concrete is a fracture phenomenon potentially dependent on the specimen geometry and scale. In this mechanical analogy, the process of microcrack onset and propagation will be described for the case of a concrete cylinder in uniaxial compression. Let's imagine that the cylinder is built of a set of Representative Elementary Volumes (RVEs), each composed of several aggregates surrounded by cement paste. Let's assume that damage occurs only at the level of the hardened cement paste (strong aggregates, representative for normal strength concrete). The behaviour of the RVE is analogously represented with a group of parallel connected chains (refer to *Fig. A3-1*b). The heterogeneities inside the RVE are represented by the fact that the chains have slightly different (yet comparable) mechanical properties. The onset of a microcrack inside the RVE is analogously represented as the loss of the weaker chains, while the other chains are subjected to a higher stress due to local stress redistributions (between points A and C in *Fig. A3-1*). With the propagation of microcracks, further chains are lost (between points C and D in *Fig. A3-1*). Here, the propagation can continue in the bulk cement paste or towards the aggregates, where it usually tends to follow the path along the paste–aggregate interface. However, as long as the microcrack onset and propagation is stable, the RVE is capable to bear the local stress redistributions on itself, and analogously the group of parallel chain elements is capable of bearing a stress portion. At the onset of tertiary creep (point D in *Fig. A3-1*), which is associated with the coalescence of well-developed microcracks, it is suggested by analogy that some of the RVEs are broken, so the stress is redistributed to the adjacent RVEs (point E in *Fig. A3-1*). That means that certain chain groups are lost and the adjacent groups are subjected to higher stress. The progress of coalescence is followed by the loss of further chain groups, until there are no more chain groups available to bear stresses (failure occurs, point F in *Fig. A3-1*).

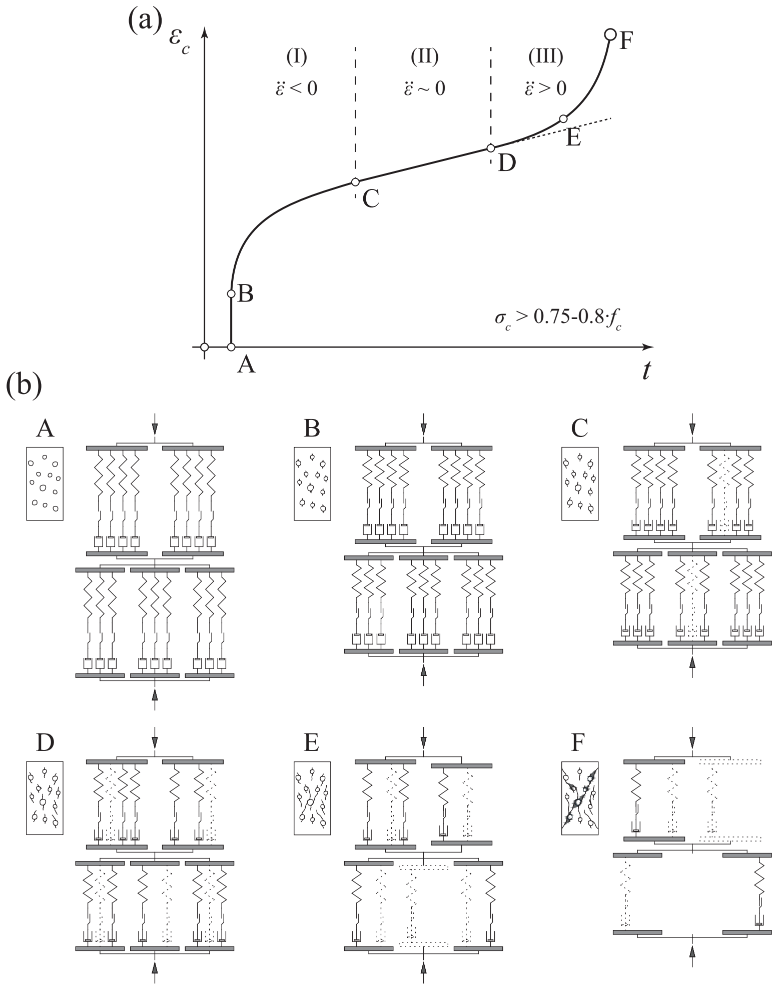

Fig. A3-1 Rheological model analogy for nonlinear creep development under high sustained loads: (a) evolution of nonlinear creep strains with time, (b) rheological representation of each stage

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Appendix A4 Envelopes of the strength reduction after a constant level of permanent stress

Section 4.4.4 presents the envelopes of the reduction of strength (points A in *Fig. 4-10*a-c) for a constant level of permanent stress (σ_{perm}) applied at t_0 = 90 days and for different time durations, on a concrete with a an early strength class parameter s = 0.25. The purpose of this appendix is to give a complete parametric study on the influence of the age of loading t_0 and the cement early strength class on the envelopes of strength reduction.

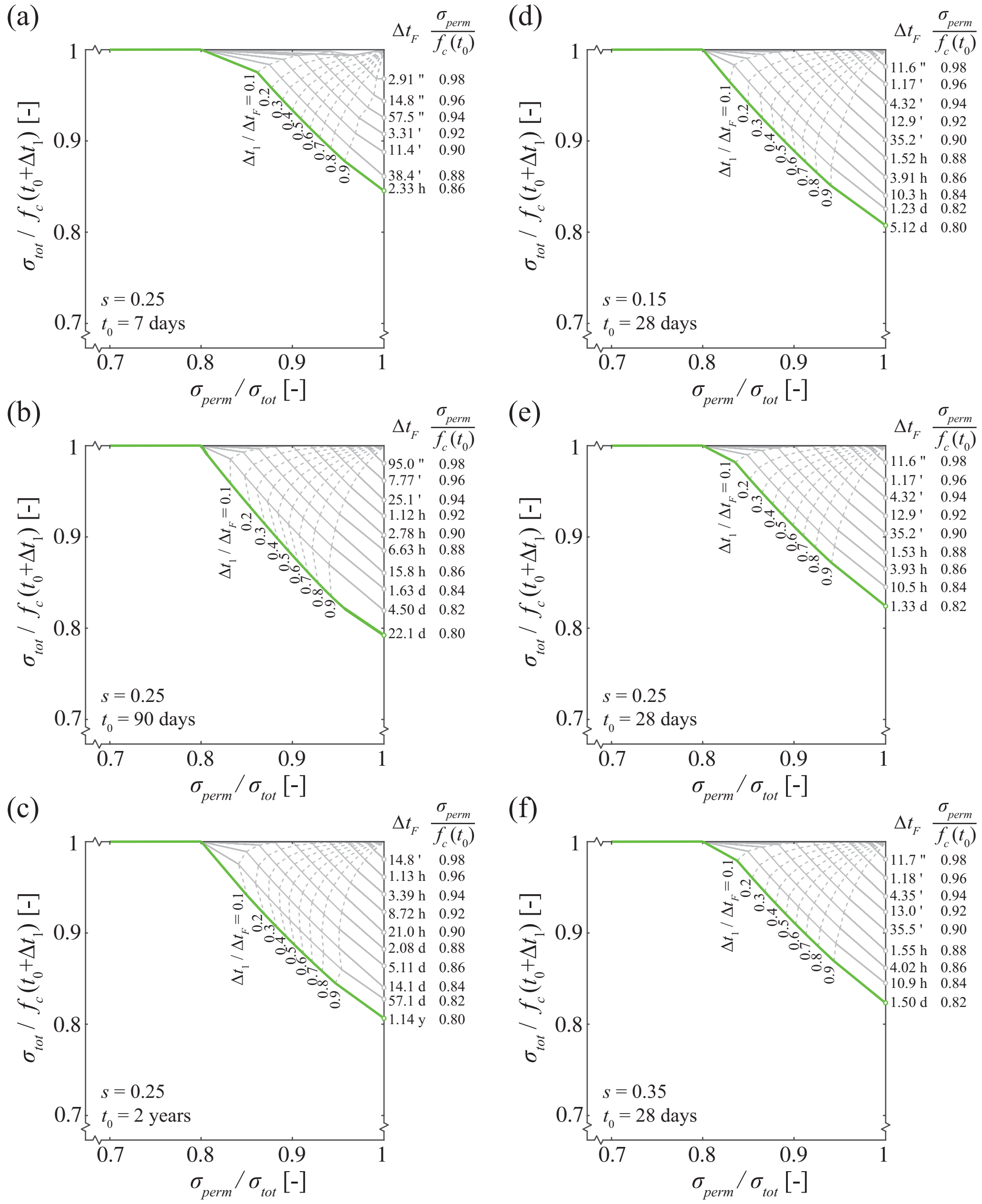

Fig. A4-1 Parametric study on the influence of (a-c) age of loading t_0; and (d-f) cement early strength class on the strength reduction after permanent actions of various magnitudes and durations

The influence of the age of loading is presented in *Fig. A4-1*a-c and the influence of the cement early strength class is presented in *Fig. A4-1*d-f. The case corresponding to the permanent stress level equal to the sustained load strength is represented by the green curve, while cases with higher levels of permanent load are plotted as grey curves.

Appendix A5 Extended parametric study on the influence of time of application of variable loads on the strength of concrete

Section 4.5.3 presents a parametric study on the influence of the duration of the application of the variable load on the concrete strength after an action of a sustained load of a determined duration Δt_1 (in this case Δt_1 = 15 days, which is the time when most of the damage due to secondary creep develops). A further parametric study presented in this appendix aims to determine the critical duration of Δt_1 for stress levels $\sigma_{perm} / f_c(t_0) < 0.8$ (point C' in *Fig. 4-10*d-f). The results are presented in *Fig. A5-1*. It can be noted that for a permanent action in the upper range of the serviceability limit state domain ($\sigma_{perm} / f_c(t_0)$ = 0.65), the reduction of strength under an additional variable action of duration of 1-2 hours is in the range of 2% (rather negligible) whereas for 1 day the reduction may amount up to 7% (which is rather non-negligible). According to this simulation, it can be admitted that variable actions up to 1 hour can be seen as no concern for the strength.

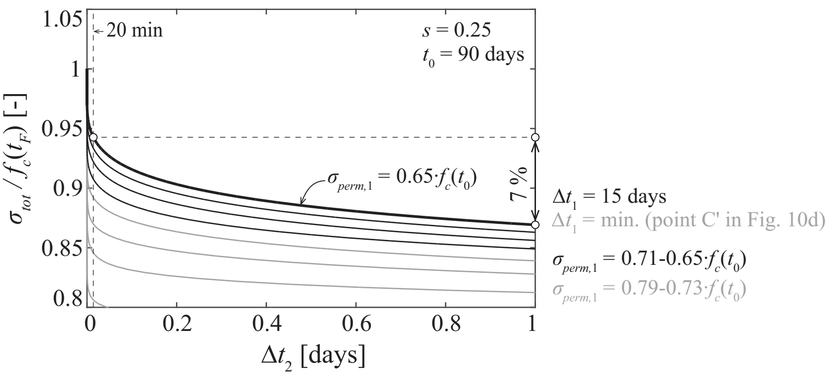

Fig. A5-1 Influence of the duration of the application of the variable load: strength reductions in the first day for different initial sustained constant stress levels.

Another aim of this appendix is to complete the parametric study of section 4.5.3 for large ages of application of the permanent action (t_0 = 20 years). *Fig. A5-2* plots a comparison for t_0 = 90 days and t_0 = 20 years, and for two values of the early strength class parameter s = 0.15 and s = 0.25 (results for s = 0.35 are almost identical to s = 0.25).

It can be concluded that if the permanent action is applied on a very old concrete, additional variable loads cause less severe strength reduction and they can be carried with no significant damage for periods of up to 10 days (for the previous case of t_0 = 90 days it was concluded that a reduction has to be accounted for variable actions of a duration longer than some hours). An extension of this parametric study is necessary to understand if variable actions of longer durations are still a concern in the assessment of old concrete structures.

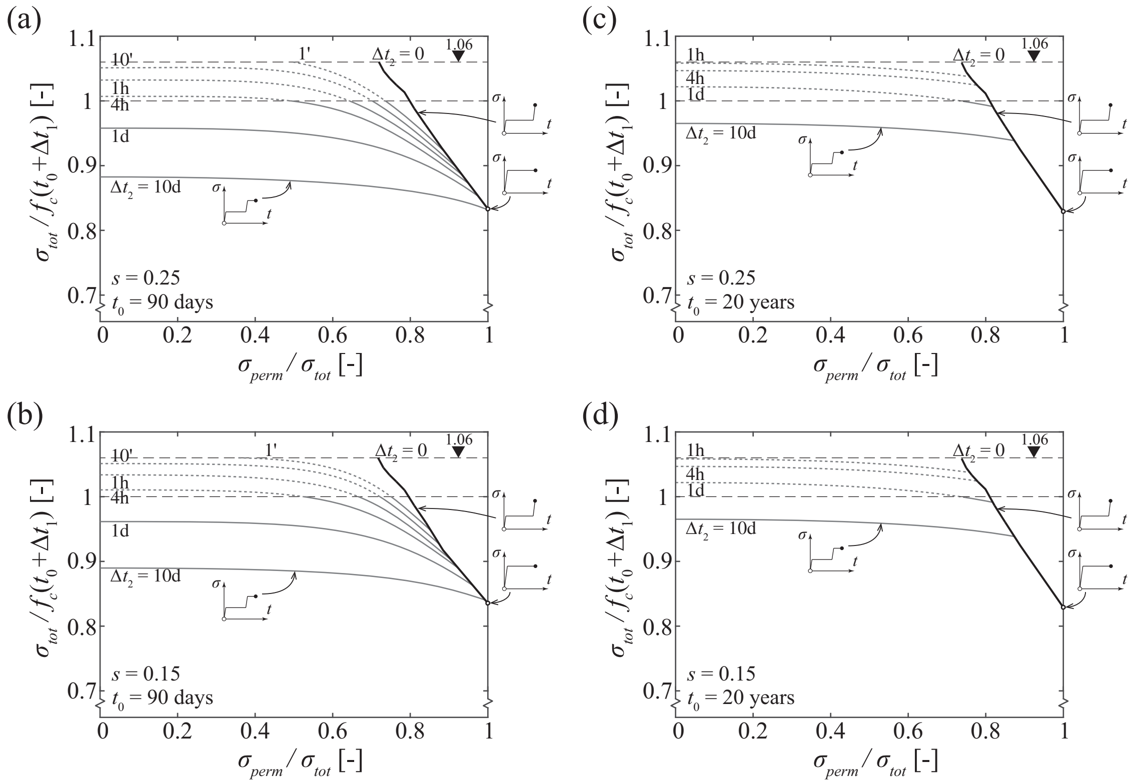

Fig. A5-2 Influence of the duration of the application of the variable load: strength reductions (for structural design) as a function of the duration of the variable action for: (a-b) t_0 = 90 days; (c-d) t_0 = 20 years.

Curriculum Vitae

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2012	Research intern, Edinburgh Napier University, *Edinburgh, Scotland*
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2018	Outstanding Paper Award, *Journal of Advanced Concrete Technology*
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Personal	Jazz and accordion, mountain lover, recreational swimmer

*for a full list of publications, see Chapter 1 of the thesis